NEW YORK STATE
DEPARTMENT OF TRANSPORTATION

GEOTECHNICAL CONCEPTS REPORT
SR 443 Delaware Avenue Landslide
Elsmere, New York

JUNE 5, 2000

LANDSLIDE TECHNOLOGY

REPORT

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EXECUTIVE SUMMARY

Landslide Technology was retained on May 20, 2000 to provide expert second opinion consultation services for the emergency response at the recent landslide along State Route 443 (Delaware Ave.) in Elsmere, New York.

The landslide became noticeably active on Tuesday May 16 2000. Two days later the landslide had retrogressed toward the highway and undermined a produce market business located on State Route 443. The outer parking lot / yards of three adjacent businesses also became impacted by the landslide. State Route 443 is a major 4-lane collector highway, normally carrying about 19,000 (AADT).

Subsurface explorations have been conducted by NYSDOT, with input from Landslide Technology. Soil samples were obtained for laboratory testing. Instruments were installed to monitor landslide movement and groundwater levels. Preliminary analyses have been performed to evaluate slope stability, causation issues, and conceptual options for landslide mitigation.

Based on our site reconnaissances and a review of historic airphotos, it is our opinion that the slope had undergone some creep / slide movement prior to the May 2000 storm. Due to the previous movement, the varved clay underlying the lower slope likely had a shear strength very close to its residual value. The upper slope probably had slightly higher shear strength due to greater overburden stress and less creep activity over the years. This is evident in the 1992 airphoto where a curved scarp is apparent about midway up the slope. When fill materials were placed near the top of the slope, the stability became slightly reduced, closer to FS=1.0, but still marginally stable. Tension cracks apparently developed in the upper slope by April 2000 (based on an airphoto), prior to the significant storm period. Therefore, it is possible that a previous storm may have caused higher groundwater conditions to initiate landslide creep. The cumulative effect of previous fill placement may also have contributed to the landslide creep. The prolonged May 2000 storm likely caused extremely elevated groundwater conditions, and may have caused scour along the stream bank due to unusually high stream levels. The recent storm also produced significant surface water runoff, which could have flowed into the tension cracks and increased hydrostatic forces acting to push the slide mass. It is likely that the historic toe slumps were activated first, followed by subsequent slumps retrogressing upslope (as evidenced by the series of slump scarps). As the landslide began to move, the shear

strength of the overconsolidated varved clay became remolded and therefore decreased to residual levels, resulting in a lower FS and unstable condition. The quick loss of resistive forces resulted in a sudden collapse of the ground, considered a 'slump earthflow', which caused the slide mass to flow into the stream channel and effectively built a debris dam. The debris dam, in turn, caused inundation of the area upstream, resulting in added water influence on the slide toe. Subsequently, additional slumps removed support from the slope and retrogressive movements occurred. The sudden collapse resulted in the rapid accumulation of soil at the toe, which created a new more stable configuration (FS>1.2), other than leaving an unstable headscarp at the top of the slope. Also, the area immediately to the east appears to still be unstable, as evidenced by continued tension crack movement. This easterly area might still undergo further movements.

Conceptual mitigation options were evaluated, with the preferred option consisting of the following methods:

- Excavation (unloading) of the upper scarp area, combined with a slope buttress. The new slope would likely need to be reinforced and / or rock-buttressed. Relocate storm drains and culverts away from landslide.
- Regrading of the slide mass to facilitate surface drainage and to remove high points.
- Counterberming and drainage of the slope toe to make the landslide less vulnerable to rapid groundwater increases. The drainage would possibly consist of about 3 or 4 French drains perpendicular to the stream channel. Other drainage systems may become necessary as new information is developed from the remaining geotechnical investigation. It is critical to slope stability that the slide debris that has accumulated over the original Normans Kill stream channel not be removed.
- Realigning and reinforcing the stream channel to allow for the landslide toe to remain in place and to extend the slope toe on the east side of the main slide. Reinforcement should consist of a riprap / buttress, with rock groins to prevent undermining of the new channel and slide toe.

Other options included a) realignment of the highway (prohibitively expensive, with significant community impacts), b) tiedback retaining wall at the headscarp (prohibitively expensive and possibly structurally impractical), c) replacement of the roadway subgrade with light-weight fill to reduce forces that could negatively affect slope stability (expensive and probably not necessary), and d) a 3H:1V sideslope

buttress on the upper slump block to laterally support the roadway (added load could cause additional slide movement and instability).

Geotechnical investigations are still in progress by NYSDOT and therefore the conclusions contained herein are conceptual and preliminary. The results of the forthcoming data will need to be evaluated and the findings of this report reconsidered as necessary.

1. INTRODUCTION

1.1 General

Landslide Technology was retained on May 20, 2000 to provide expert second opinion consultation services for the emergency response at the recent landslide along State Route 443 (Delaware Ave.) in Elsmere, New York. This letter summarizes our emergency evaluations and presents interpretations of landslide mechanisms and preliminary comparison of mitigation concepts.

1.2 Background Information

The landslide became noticeably active on Tuesday May 16 2000. Two days later the landslide had retrogressed towards the highway and undermined a produce market business located on State Route 443. The outer parking lots yards of three adjacent businesses also became impacted by the landslide. State Route 443 is a major 4-lane collector highway, normally carrying about 19,000 (AADT). This highway is a main link for suburban communities to the state capitol Albany. The landslide affects the DOT, several businesses, a group of residences, the Town of Bethlehem, the City of Albany (particularly it's 48-inch water main supplying approximately 100,000 users), emergency providers, and road users. The NYSDOT is charged to develop solutions to restore the road as soon as possible.

1.3 Scope of Work

The following workscope was performed:

1. Visit the site on May 20th and perform an initial reconnaissance.
2. Meet with NYSDOT staff to obtain background information and objectives for the second opinion services, and review landslide photos and site data.
3. Perform evaluations of possible landslide mechanisms, and perform initial parametric stability analyses.
4. Develop preliminary opinions regarding field explorations & testing, and conceptual solutions.
5. Attend a second meeting with NYSDOT staff on May 25th to discuss exploration and testing recommendations, initial opinions of potential landslide failure mechanisms, and concepts for mitigation. Obtain additional data and references from the NYSDOT.

6. Evaluate data obtained in subsurface explorations, instrumentation, and testing performed to date.
7. Research relevant information regarding shear strengths of varved clays and possible groundwater sources.
8. Perform conceptual stability analyses based on new data, references, and correlations.
9. Review the preliminary opinions regarding landslide mechanisms, explorations & testing, and analysis methods / considerations.
10. Develop one to three feasible conceptual solutions, including the relative advantages and disadvantages, development and construction time, relative risk, and opinions of relative construction cost.
11. Prepare a report summarizing opinions on landslide mechanisms and preliminary options studies.

2. SITE RECONNAISSANCES AND OFFICE REVIEW

2.1 Reconnaissance and Office Review

The initial reconnaissance of the landslide was performed on May 20, 2000. The features of the landslide were observed, including the headscarp / roadway area and adjacent slumps, the side scarps down to the Normans Kill stream channel, slump blocks, tension cracks, hummocky ground surface and sag ponds, extended slope failure toe, relocated stream channel, and slopes adjacent to the landslide.

Geotechnical issues affecting emergency road closure and potential slide impacts to the highway and nearby buildings were discussed at the site with NYSDOT personnel. The risk to upslope property was evaluated by extending an imaginary 3H:1V and 4H:1V sloped lines from the toe of the headscarp to identify which buildings could be at risk.

Photographs taken the first few days following the landslide collapse were reviewed. NYSDOT's initial exploration program and initial cross-section modeling of the landslide were described. The office meeting was held to review the model cross section and background topographic information, as well as to identify 'next steps' of the geotechnical investigation.

A return visit to the landslide was performed at the end of the day (May 20th), primarily to reconnoiter the active landslide mass. This was done to gain further insights on the relative activity of the main slide mass, to look for groundwater seepage (springs), and to look for possible signs of pending slide enlargement. The headscarp area was visibly active, with slabs of varved clay (Albany Clay) sloughing off several times an hour. The front part of the produce market was still in place (most of this facility had reportedly collapsed on Thursday May 18th following intense rainstorms on Wednesday). The lower slide mass appeared to be either stationary or creeping undetectably. The central and toe areas were traversed to determine the relative consistency of the slide mass and to take a closer look at the distorted ground surface. For the most part, the clay slide mass was firm with localized soft or ponded areas. The clay surface was locally slippery because of its plasticity, moisture content, shiny varved surfaces, zones of remolded clay, and localized area of surface water and wetting from recent precipitation.

The lower slide toe (debris dam) appeared to have thrust up against the original north stream bank and pushed debris onto the north-side field, covering bank vegetation and surrounding several trees. On May 20th, the slide toe was viewed from the north side of the relocated stream channel. It appeared that the clays were eroding slowly from the stream flows (although some chunks had broken back) and that there were no perceptible signs of active slide movement. The new relocated south stream bank exposed the apparently undisturbed native clay up to about elevation 100 feet, mantled with slide debris about 5 to 15 feet in thickness. The stream water level appeared to be low. During the three half-hour observations in the toe area, there was no apparent sloughing into the stream, shifting of soil blocks, or sounds of trees moving or cracking. These observations lead to the opinion that the lower landslide mass had probably come to rest shortly after the major sudden slope failure.

Wet ground and springs were evident on both the adjacent east and west 'unfailed' slopes, from the stream bank up to about elevation 130 feet (visibly estimated and checked on the pre-slide cross section). In these areas, there were signs of ancient / historic slide slumps and scarps that had become mostly overgrown with vegetation. Some fresh tension cracks / rotational slumps were identified beyond the east side of the main landslide. The slide toe had not dammed the stream channel at this location and therefore this slide area did not have as much toe support. Our field interpretation was that the landslide might enlarge on the east side and could affect adjoining property, utilities, and the auto maintenance facility upslope.

Review of the site conditions, topographic maps, and airphotos was useful to estimate relative filling near the top of the slope, close to the highway. It appeared that the ground beneath the produce market / fruit stand / parking area was on fill that had been placed a small amount at a time over many years. The fill consisted of a variety of materials, including soil, woody debris and other waste materials. Reportedly, this fill had experienced minor incidences of subsidence and movement that appeared to be localized to the fill itself.

The auto maintenance facility also appears to have filled on its northerly side. A gabion wall supports part of this fill. Following the major landslide activity, part of the westerly fill / ground slope had been excavated (unloaded) as a preventative measure by the owner. Some fill material appears to have been side-cast onto the slope to the northeast of the auto maintenance building, possibly adding weight locally to the slope.

The medical office building and lot to the west of the slide headscarp appears to have been constructed many years ago. Its paved area appears to have skirted the swale-shaped landslide area, with minimal fill (if any) placed close to the slope.

The headscarp was observed for possible signs of seepage. No signs of seepage were evident. The upper portion of the headscarp appears very steep, ranging from 1H:2V to near-vertical. Extensive raveling and shallow slumping was observed in the Albany Clay exposed in the headscarp face. Most of the slide cracks appeared to be circular slumps, with the upslope end rotated downward. There were no evident graben features, although some compression mounds were observed.

On May 20th, exploration and testing goals were discussed. The need for including instrumented borings within the slide mass was raised. The data to be collected included groundwater levels, soil shear strengths, depths of slide movement, soil classification and index tests, and anisotropic effects.

2.2 Initial Evaluation of Landslide Mechanisms

Based on the foregoing information, an initial evaluation was made of possible failure mechanisms. Parametric stability analyses were performed to test the initial failure mechanism options and to determine the sensitivity of the shear strength and groundwater assumptions.

The failure mechanism model needed to account for the very steep and high headscarp, therefore the shallow circular failure mode was ruled out. Two potential failure mechanisms appeared possible: 1) a translational slide following the horizontally-bedded clay varves, and 2) a deep-seated circular failure plane. Both failure mechanisms account for the steep headscarp. However, the toe conditions would be different. For the translational slide mechanism, the toe would have compression and thrust features with a nearly horizontal failure plane exit angle. The deep circular failure plane would exit upwards steeply and would cause heaving and tension cracks at the toe, with excess material spreading across the north bank of the stream channel.

The initial stability analyses reflected the foregoing conditions. Initial assumptions of shear strength included: 1) a range of undrained shear strength from 500 psf to 2000 psf, 2) effective stress phi angle of 10 to 20 degrees, with effective cohesive intercepts of 0 to 400 psf.

These initial analyses indicated that both failure mechanisms were possible, with Factors of Safety, FS, for the slide failure condition close to 1.0. The initial analyses also indicated that the post-failure condition could have a FS substantially greater than 1.0 because of the slide mass build-up at the toe of the slope.

Consequently, on Monday May 22nd, we provided preliminary interpretations, suggested revisions to the Geotechnical investigation program, and a range of possible mitigation measures.

- The borings and instrumentation should extend down to the shale unit in the event of a deep-seated circular failure plane.
- The borings within the landslide area should be located between the middle and original stream channel, where a deep-seated circular failure plane is likely to be deepest.
- Identify any aquifers and artesian zones.
- Immediately establish a survey monitoring program to determine if the central and lower portions of the landslide have stopped moving.
- Survey a cross section from the highway to the flood plain north of the relocated stream channel, down along the 'gut' of the landslide.
- Perform undrained shear strength tests on remolded clay / silt samples and on selectively trimmed clay–only varves (remolded).

- Perform drained direct shear tests to determine effective residual shear strength of the clay / silt mix and clay-only remolded specimens.
- Classification tests should include Atterberg Limits to identify the level of plasticity and for use in correlations with typical shear strength values.
- Possible mitigation options that were evaluated include:
    - o Keep the slide toe mass in-place and do not restore the pre-slide stream channel (i.e. utilize the slumped toe mass as a counterberm).
    - o Place a fill buttress against the oversteepened headscarp to provide lateral support to the highway. This might include light-weight fill materials to reduce driving forces at the top of the landslide.
    - o Unloading part of the roadway embankment and replacing it with light-weight extruded polystyrene might be a benefit if the potential for the landslide to retrogress further is high.
    - o Structure and tie-backed wall solutions to stabilize the landslide appear to be impractical due to poor clay conditions, significant depths involved, and high costs.
    - o Road realignment was briefly considered, but dismissed because it would create significant new impacts and would likely cost much more than the options to stabilize the landslide.

2.3 Second Reconnaissance and Meetings, May 25th

Meetings were held at the NYSDOT to discuss: 1) recent landslide observations; 2) the progress of exploration drilling, sampling, and instrumentation; 3) initial laboratory program; 4) Normans Kill stream channel issues; 5) initial stability analysis results and implications; 6) geologic and engineering data references; 7) airphotos and surveys; and 8) concepts for landslide mitigation and roadway protection. In addition, a second reconnaissance was made to determine the preferred locations for borings within the landslide mass and the objectives for sampling and instrumentation. The central and toe area of the landslide were traversed, as well as the slumped area at the toe of the headscarp and the headscarp itself.

The landslide mass appeared to be 'not moving'. The headscarp has experienced occasional (less frequent) sloughing and is building a clay talus slope buttress at its toe. The headscarp area was traversed during the reconnaissance. A small trail of flowing surface water was traced back up the slope to a damaged culvert

on the southeast, near the western edge of the auto maintenance property. No signs of groundwater seepage were evident in the headscarp area. Several sag ponds and wet swales were identified east of the landslide and a short distance downslope of the auto facility. The central area of the landslide contained an accumulation of logs, stumps, wood fiber, and miscellaneous construction debris, which likely originated from the fill behind the produce market.

The exploration drilling had progressed to the easterly location upslope of the headscarp. The first two borings had encountered shale bedrock at about 180 feet deep (about elevation 20 feet). The first boring on the west side had encountered noticeably softer materials below 100 feet deep. This correlates well with the headscarp height of 70 feet, indicating that there was soft material 30 feet and deeper below the headscarp.

It was agreed that the two exploration locations within the landslide mass would be sampled with Shelby tubes every 5 to 10 feet, below a depth of 30 feet, down to the shale bedrock. The inclinometer casing should be as large a diameter as can reasonably fit within the hole made with the portable drill rig. The observation wells would measure the average condition in the soils above the shale, up to elevation 100 feet. If artesian water pressures were encountered, then special piezometers might need to be installed. A boring is also scheduled for the north side of the relocated stream to correlate stratigraphy and the depths to bedrock.

The laboratory testing program was described and some new suggestions made, particularly for shear strength test normal stresses and for prioritizing the tests to be performed. The most critical shear strengths are the undrained shear (total stress) at remolded / residual conditions. Care should be taken to prevent unreasonably low values due to test and specimen deficiencies.

We were told that the stream relocation could be permitted, given the natural rain-induced landslide event. The landslide and stream relocation are natures way of creating more stable conditions. The slide mass build-up at the slope toe is beneficial to providing overall stability.

The two possible failure mechanisms were discussed (translational and deep circular). Exploration and engineering analyses are scoped to evaluate both possibilities. The initial analyses demonstrate the instability of the slope toe, which

has experienced historic slumping. The initial analyses also demonstrate the historic marginal stability of the overall slope.

Several publications were provided by NYSDOT, including reports on the glacial geology of the Albany vicinity and manuals summarizing shear strengths and stability analyses in varved clay deposits. Airphotos and the results of a cross section survey were also provided.

The concepts for landslide mitigation were discussed. The primary method near the highway is to provide a buttress fill against the headscarp, combined with the excavating back to the highway curb. It was mentioned that the highway width could be reduced from 4 lanes to 3, if absolutely necessary. The goal for buttress placement is to limit fill onto the landslide to avoid lowering the FS below 1.3. To do this reliably would require accurate peak and residual shear strength determinations (along and across clay varves). Counterberming of the landslide toe and providing French drains are considered reasonable techniques at the toe. Other methods may become necessary, such as the use of light-weight fills, deep relief drains, and stone columns. Geotextiles or geogrids could be used to reinforce fill placed either at the top or toe of the landslide.

2.4 Summary of Conditions and Interpretations of the Landslide

The major landslide reportedly was first noticed at about 2:30 PM Tuesday May 16th, as the ground behind the produce market was observed to break off. A second slump collapsed about an hour later, causing a sudden and large movement of the landslide mass that pushed the south stream bank northward and filled-in the stream channel over a distance of about 200 to 300 feet. This effectively dammed the Normans Kill stream and cause stream water to rise and widen. Prior to the landslide dam, the stream channel was reported to be about 90 feet wide. The rising reservoir caused by the landslide dam affected upstream properties and exerted additional water pressures on the slide mass. The headscarp was approximately 70 feet high on May 17th prior to collapse of the fruit stand building. The slope of the headscarp was estimated to be about 1H:2V, from review of the photographs. By May 19th, continued sloughing of the steep headscarp had retrogressed about 20 feet, resulting in collapsed material filling the upper slump. The NYSDOT closed the highway after the headscarp continued to actively retrogress. The City of Albany hired a contractor to excavate a new stream channel to allow passage of the stream water around the north of the slide dam to protect their 48-inch water main. The water main is the primary

supply of water from the Alcove Reservoir to the entire City of Albany. By Saturday May 20th, the head scarp was less than 50 feet tall because of the accumulation of slough at its toe. By Thursday May 25th, the slough had covered approximately half of the headscarp height. The landslide has not noticeably affected the new stream channel, which has been partially protected against stream erosion with riprap rock on its north bank. The landslide features and buried original Normans Kill channel are shown on the Oblique Photograph, Figure 3, and on the Cross Section, Figure 4.

The landslide occurred suddenly following a prolonged period of precipitation and high stream flows/levels. There were reports that the swollen stream had eroded the toe of the hillside. The slide became evident when the large headscarp reached up the slope to the rear of the fruit stand building. The wet weather conditions caused significant problems and damage within 13 counties. Scattered thunderstorms and hailstorms knocked down trees and power lines, and caused damage to buildings and flooding of some roads.

The sloping ground on both sides of the landslide was reconnoitered on Saturday May 20th. The lower third of the slopes appeared very wet from groundwater discharge (springs). A series of historic and active slump scarps were evident in the wet slope areas. The surface features consist of a series of circular slumps. There were no signs of active movement occurring in the middle and toe areas of the landslide mass. Therefore, the noticeably active movement appeared to be localized to the headscarp area. Figure 1 presents a Site Plan of the landslide and approximate slide limits and features.

The landslide mechanism could be either a near-horizontally moving block on weak varved clay or a deep-seated circular slide, as shown on the Subsurface Conditions Cross Section, Figure 5. The depth of the landslide from the highway elevation is deeper than the highest headscarp observed (70 feet), and is likely in the order of 90 to 120 feet deep. Since the landslide occurred rapidly and that the main slide mass is apparently no longer moving, it is our opinion that the slide has overcompensated (built a new larger toe) and has achieved a higher level of stability. Analyses would need to be performed to estimate the degree of stability achieved.

The evidence of the historic scarps and slumps in the lower slope verifies the susceptibility of the lower wet slope to erosion and slumping. It is reasonable to interpret that the slide was initiated at the toe of the hillside and progressed upwards as the ground support was lost. The primary causation appears to be a direct result of

unusually wet conditions. Surface water from prolonged precipitation likely entered and filled open tension cracks, which increased driving forces and water pressures within the slide mass. The reservoir caused by the landslide dam further exacerbated the high groundwater conditions and further destabilized the slope toe. Other factors that probably affected instability include: weak clay strength, high groundwater conditions and recharge, fill placement at the upper area of the slope (next to the highway). It appears the hillside was marginally stable prior to the storm and also before fill was placed on the slope. This is consistent with geologic reports on local slope failures in the Albany Clay unit, particularly when the clay slopes are higher than 40 feet and steeper than 3H:1V.

3. FIELD AND LABORATORY INVESTIGATIONS

By Friday May 19th, the NYSDOT drill crew had started the boring on the west side above the head scarp. Three borings were eventually drilled above the head scarp: one each on the east and west sides of the scarp and one south of the main scarp (along the approximate centerline of the slide). Each boring location was drilled to bedrock (shale) and a slope inclinometer casing was installed to measure ground movements. NYSDOT and Landslide Technology further developed the details of the exploration and instrumentation program jointly.

Two observation wells were installed above the headscarp. The location of the observation wells was close to the central upper inclinometer boring. The depths of observation wells P2A and P2B were 150 and 80 feet below highway grade, respectively, where the upper seals were placed at depths of 140 and 70 feet below highway grade. The water level in P2B is currently at elevation 174 feet, about 29 feet below highway grade. P2A is recording deeper groundwater conditions. Water level measurements are shown in Appendix A. We have recommended two vibrating wire piezometers be installed near the headscarp, at depths of 50 and 90 feet below highway grade, to verify potentially high groundwater pressures.

Two exploration locations were established within the translated slide mass. The drill rig was mobilized by May 28th. At each location, the soil profile was to be sampled and inclinometer casings installed to bedrock. Adjacent to both inclinometer borings, vibrating wire piezometers were to be installed to measure groundwater pressures in the estimated vicinity of both estimated slide plane options. Shelby tube samples are to be obtained through the lower half of the slide mass in order to obtain samples for shear strength testing and classification tests. We understand that Boring

for SI 142D has been drilled and sampled; however, the installation of inclinometer casing has been postponed for a few days until headscarp movements become less threatening to the drill crew. To help decrease the risk, the headscarp is being excavated back.

The logs of borings for the first three borings above the headscarp were reviewed. Laboratory testing is underway, and initial results have been reviewed. The measurements made to date in the inclinometers and piezometers as well as survey prisms are shown in Appendix A. The appendix also presents a summary of the first phase of laboratory testing.

The materials generally consist of clay and silt varves of the Albany Clays unit, underlain by occasional sand and gravel seams, which in turn, are underlain by a relatively thin mantle of clayey gravel glacial till on top of shale bedrock. The varved clay has a Plasticity Index that can reach about 15% to 25%, and includes the clay mineral septachlorite. Shear strengths in the varved clay behave anisotropically because of the glacial stress history producing an overconsolidation ratio of about 4. Therefore, the shear strength of remolded clay can be substantially lower than the peak strength. The Albany Clays contain thin sand seams, but are generally considered poorly draining. The underlying sand was deposited from glacial outwash events. The clay and silt varves were formed in a glacial lake environment, thus the geologic term 'glaciolacustrine' that is used to describe the origin of such soils. The varves are usually less than inch thick (varves about 0.3 to 0.5 inches were observed in exposed blocks of the slide mass).

4. PRELIMINARY EVALUATION OF LANDSLIDE MECHANISMS

The following conditions were evaluated to assess slope behavior and to confirm use of reasonable parameters:

- Toe stability during elevated surface water and groundwater conditions,
- The impact of fill placement on slope stability,
- The initiation of the landslide,
- Stability achieved by the rapid slide movement and toe-building.

The stability analyses were performed using XSTABL software. Both total stress and effective stress analyses were used to model rapid, initial failure and longer-term partially drained conditions. The landslide topography was developed from existing topographic maps and a survey performed by NYSDOT the week following the start of the enlarged landslide. Groundwater conditions were based on groundwater seepage elevations and initial samples from the first boring. Subsequent observation wells and piezometers were used to refine groundwater assumptions. Shear strengths of the soils were initially estimated pending laboratory testing. Several references were consulted to evaluate the reasonableness of strength estimates (refer to Section 7). The assumed conditions and soil parameters are shown on the interpreted geologic cross section, Figure 5. For example, the undrained shear strength for residual conditions along clay varves was estimated to be 600 psf; and 1000 psf across the varves. Also, intact (peak) strengths of 2500 to 3000 psf across varves were conservatively estimated. Estimated effective stress parameters ranged from a friction angle of 18 to 20 degrees, plus a cohesive intercept value of 0 to 400 psf. The foregoing values rely on various assumptions, such as the OCR (the Over-Consolidation Ratio, maximum past pressure), groundwater levels & effective vertical stresses, clay mineralogy, plasticity, etc.

Appendix B presents the preliminary analysis results. The analyses show that the hillside toe was essentially unstable and verified the historic incidences of lower slope ground movement. The overall slope, from the stream up to the highway, was 'marginally stable', and not as vulnerable as the slope toe. The placement of fill material on the slope and the parking area and facilities reduced the stability of the slope to the low end of 'marginal stability' levels, with the computed factor of safety, FS, slightly above 1.0. Erosion and slumping of the slope toe would have reduced the stability of the overall slope. Elevated and imbalanced water levels would have caused a decrease in slope stability. Once movement occurs, the clay soils undergo remolding and stress changes, resulting in significantly reduced shear strengths (approaching residual levels).

The strength along the varves of the clay is typically lower than across the varves. Therefore, lateral movement along clay varves can be a critical condition. Two types of failure surface were investigated: 1) translational block, and 2) deep circular. Both analysis types had to demonstrate they could produce the steep and high headscarp. Therefore shallow circular failure surfaces were considered inconsistent for the overall landslide condition.

After the landslide came to rest, the analyses indicate the factor of safety, FS, could be in excess of 1.2 for total stress (short-term) conditions long-term effective stress conditions. The toe of the slope now extends an additional 200 feet to the north, which provides additional toe resistance. The estimated shear strengths and groundwater conditions should be verified by field and laboratory testing to confirm (or revise) the estimated stability increase.

- The lateral slide movement case, along clay varves, can produce the type of features evident at the subject slide.
- The deep circular slide mechanism is still possible, but maybe not as likely as the lateral slide mechanism.
- The shear strengths that were calculated using the reference publications result in reasonable factors of safety.
- The existing post-slide conditions might have a FS of at least 1.2 based on the conceptual analyses (both in the total stress and effective stress cases). While groundwater lowering and toe counterberming (on top of the existing slide toe) can be beneficial, they provide only small increases in calculated FS for the short-term total stress case. The benefits would be greater for the long-term effective stress case. Counterberms that lengthen the slide toe (push the stream further north) would increase the FS for the short-term total stress case.

Aerial photographs were interpreted for evidence of landslide conditions. The 1986 photographs have been observed in stereographic projection and show landslide evidence including hummocky topography and arcuate-shaped benches, which "step" down the slope towards the river and appear to be slump blocks, as illustrated on Figure 1.

The 1992 aerial photographs that were observed do not cover the slide area in stereographic projection. However, arcuate-shaped shadows on the ground surface appear to be active ground distress features. At the base of the fill behind the produce market building are shadows that appear to be bulging ground that could represent the toe of a slide in the area of the fill. About half the distance between the river and the top of the slope is a series of shadows that appear to be a slide scarp that steps down towards the river. Also, the Normans Kill channel narrows slightly about a hundred feet downstream from the apex of the sharp left-hand river bend, and down slope of the apparent scarp. The arcuate-shaped shadows and narrowing of the river

may be evidence of two areas of sliding: an upper slide within or beneath the fill, and a lower slide moving into the Normans Kill channel.

The April 2000 aerial photograph shows apparent distress on the top of the fill behind the produce market building, as shown on Figure 2. Arcuate-shaped shadows in the surface of the fill have a shape and appearance of a series of tension cracks. Also, the south bank of the river channel appears to be slightly bowed into the river, which may be due to the landslide pushing into the river channel.

5. CONCEPTUAL MITIGATION OPTIONS

After the landslide mechanisms were studied, various landslide mitigation options were evaluated and compared. The increased FS resulting from the toe-building of the slide is about the value desired for landslide mitigation (FS>1.2). The highway is unsupported on the north side by the oversteepened headscarp. Flattening the upper slope to a stable level can best mitigate this condition. The cost-effective solution appears to be trimming the upper scarp slope to a flatter slope, set-back from the westbound highway curb about 5 feet (and not having to give up a travel lane). It appears possible that no fill would be needed to buttress the scarp since the cut slope would likely fit with ‘excavation only’. A 2.5H:1V cut slope might work because the height of the cut would be less than 30 feet and the groundwater is likely to be deeper than the toe of the cut. However, previous experience suggests that the slope be at least 3H:1V in these clay materials to obtain trouble-free performance.

A variation of the cut slope solution is to incorporate a slight amount of rock fill at the base of the headscarp and a rock blanket inlay on the slope. The slope angle can be steepened by overexcavating next to the highway and replacing with free-draining angular well-graded rock fill. The final solution should try to minimize the amount of fill placed on top of the landslide mass.

The landslide slope toe area should be protected with riprap to prevent future toe erosion by stream flows. If the riprap is made relatively thick (15 to 25 feet) and founded below streambed level, the toe resistance can be increased, which would increase the overall FS. French drains placed perpendicular to the stream and incorporated with the riprap design can help to prevent the buildup of groundwater pressure within the slide mass during future storm events, thereby reducing the likelihood of additional failures.

The middle portion of the landslide should be regraded to reduce high jutting areas that decrease stability and to promote drainage of surface water away from the

landslide. The storm drainage from the highway should be routed away from the landslide or carried across the slide mass in a continuous and tight pipe. A culvert was observed discharging water onto the slide mass from the southeast corner, and should be mitigated.

Stability analyses demonstrating the key options are presented in Appendix B. Figure 6 shows details of the conceptual options.

6. PRELIMINARY RECOMMENDATIONS

6.1 Recommendations for Investigations

The field and laboratory investigations are fundamental to validating initial assumptions and preliminary conclusions / opinions, and to provide a rational basis for design of permanent repairs and mitigations.

The existing field program consists of a series of slope inclinometers and observation wells/piezometers, as well as samples for laboratory testing. The lab tests include direct shear tests (peak and remolded specimens), the isotropically, consolidated undrained (CIU) triaxial shear tests, consolidation tests, Atterberg Limits, water contents, and unit weights.

The three major items to be confirmed are a) groundwater levels, b) peak and residual (remolded) shear strengths (both total stress and effective stress), and c) the depth of basal failure plane movement. The total stress (undrained shear) appears the more critical based on the results of the conceptual stability analyses.

Remolded/residual shear strengths can be determined by performing vane shear tests, UU triaxial tests, CIU triaxial tests, and direct shear tests. The direct shear and vane tests provide results that are more consistent with in-situ strengths of varved clays, whereas triaxial tests can cause the specimen to fail across the varves, which would over-predict the strength in the weaker, laminated direction.

<u>Suggested Additions to Field / Lab Program:</u>

1. Perform field vane shear tests (peak and residual) in a deep boring above the head scarp, from a depth of 100 feet to 170 feet, in 5-foot intervals. The FHWA

publication on embankments over varved clay (Ladd) recommends this procedure.

2. Also, perform lab vane tests on Shelby tube samples (exposed ends) to get a series of undrained shear test results. Many of these tests can be performed at low cost. It would be valuable to have these lab vane test results as an inexpensive back-up.

3. Obtain soil samples below elevation 80 feet, to coincide with the probable failure plane locations. Check the samples for possible sand seams (groundwater passage).

4. Consolidation tests: it would be beneficial to calculate the 'max past pressures' and compare with existing effective overburden pressure (assuming a groundwater table at 10 to 25-foot depth). The OCR can thus be calculated and subsequently used with Ladd's shear strength correlations.

5. Since the varved clay is likely to have an OCR of about 3 to 4, it would be preferable to adjust the confining pressure for the triaxial test higher loading to match the anticipated 'max past pressures'. The lower and middle levels of confining pressure should be estimated based on existing average overburden pressures.

6. The recommended observation wells / piezometers include:

- Boring above the head scarp – observation well screen section from 70' to 80' depth below highway grade.
- Boring above the head scarp – observation well screen section 140' to 150' depth below highway grade.
- Boring above the head scarp – vibrating wire piezometer about 50' depth below highway grade.
- Boring above the head scarp – vibrating wire piezometer about 90' depth below highway grade.
- Two borings within the active landslide – two vibrating wire piezometers at each location: one at about 30' depth (about 100' below highway grade) and the other at the lower end of the varved clay unit.
- Add a unique observation well if artesian water is encountered during drilling, to measure only the artesian layer.

7. Check existing water line and sewer line for leaks.

6.2 Recommendations for Final Design Phase

- Obtain field and lab data to support the analysis cases relevant for final design, as described in the foregoing section.
- Perform final stability analyses based on the new test and instrumentation data, as well as accepted material correlations.
- Stability analyses should be performed to verify what the steepest cut slope should be for the conditions at the headscarp.
- Obtain permits to keep the stream channel in its new location, or possibly relocated further north.
- Develop a design for the upper slope that does not add more load to the landslide mass.
- Design the regrading of the central portion of the landslide mass, to balance weight across the slide to maximize stability and to readily shed surface water.
- Design the slide toe riprap / buttress and French drain system to provide substantial toe resistance and to prevent stream erosion from behind (end-around case), and possibly add a small counterberm above the landslide toe.
- Obtain the produce market property for highway right-of-way, to prevent further development on the landslide prone ground. The auto maintenance facility might also be considered if the landslide widens to the east.
- Revegetate the slope with hydrophilic plants to draw moisture away from the slope.

LANDSLIDE TECHNOLOGY

By George Machan

George Machan
Senior Associate Engineer

7. REFERENCES

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- Airphoto 117-13; SR 443 Flight #1090.08; 1992.

- Airphoto by Col-East; #19692-6-28; April 14, 2000.

- Oblique Photographs of Landslide; May 17, 18, & 19, 2000.

- NYSDOT; Preliminary Exploration borings, Instrumentation results, and Testing results

Limitations in the Use and Interpretation of This Geotechnical Report

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

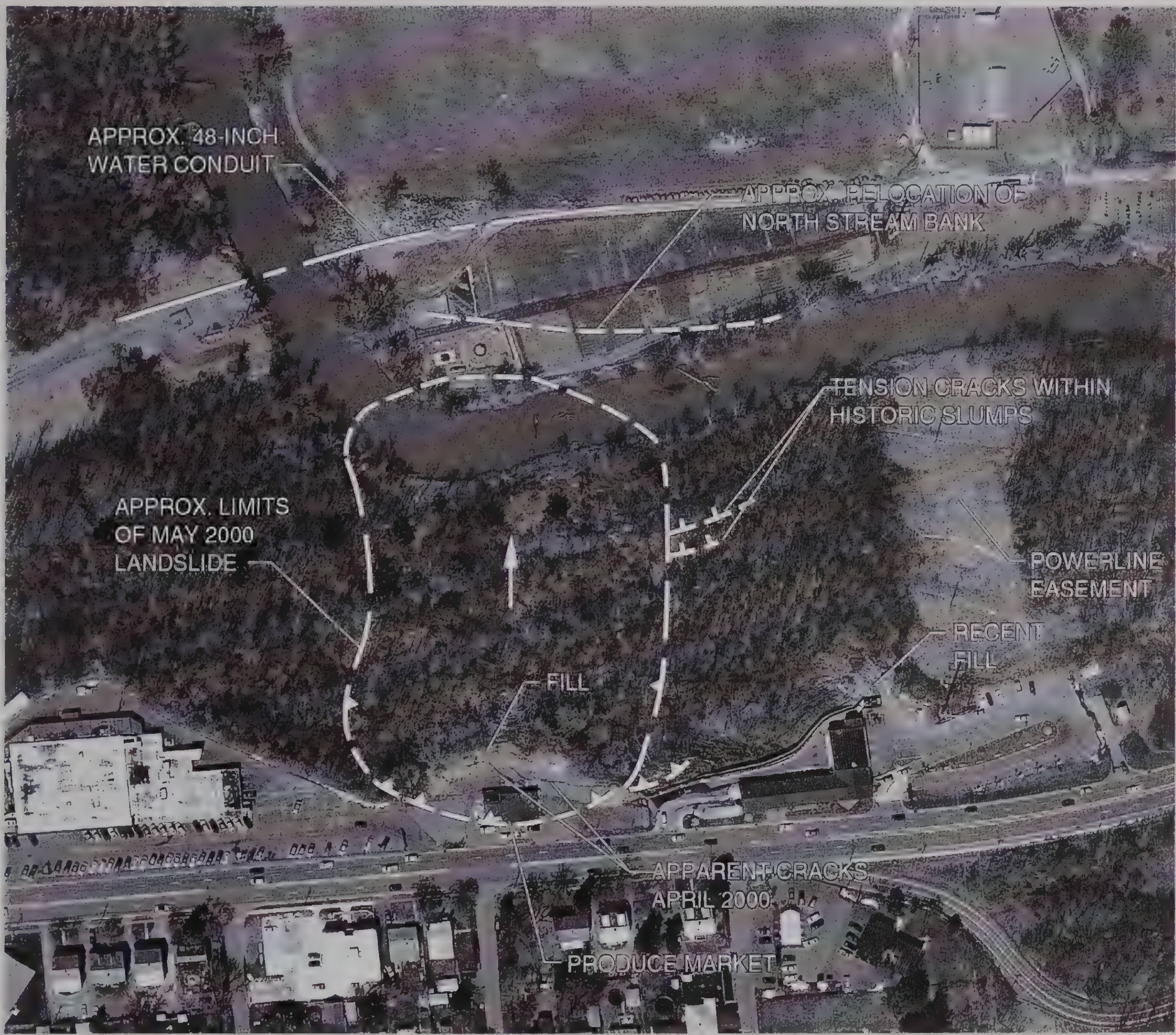
Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.

1280\FIGURE01 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE VICINITY MAP AND 1986 AIRPHOTO INTERPRETATION	DATE JUN 2000
		JOB NO. 1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 1



NOTE: LOCATIONS OF LANDSLIDE FEATURES ARE APPOXIMATE AND BASED ON MAY 2000 RECONNAISSANCE

AIRPHOTO: APRIL 18, 2000 BY COL-EAST

1280\FIGURE02 LJW

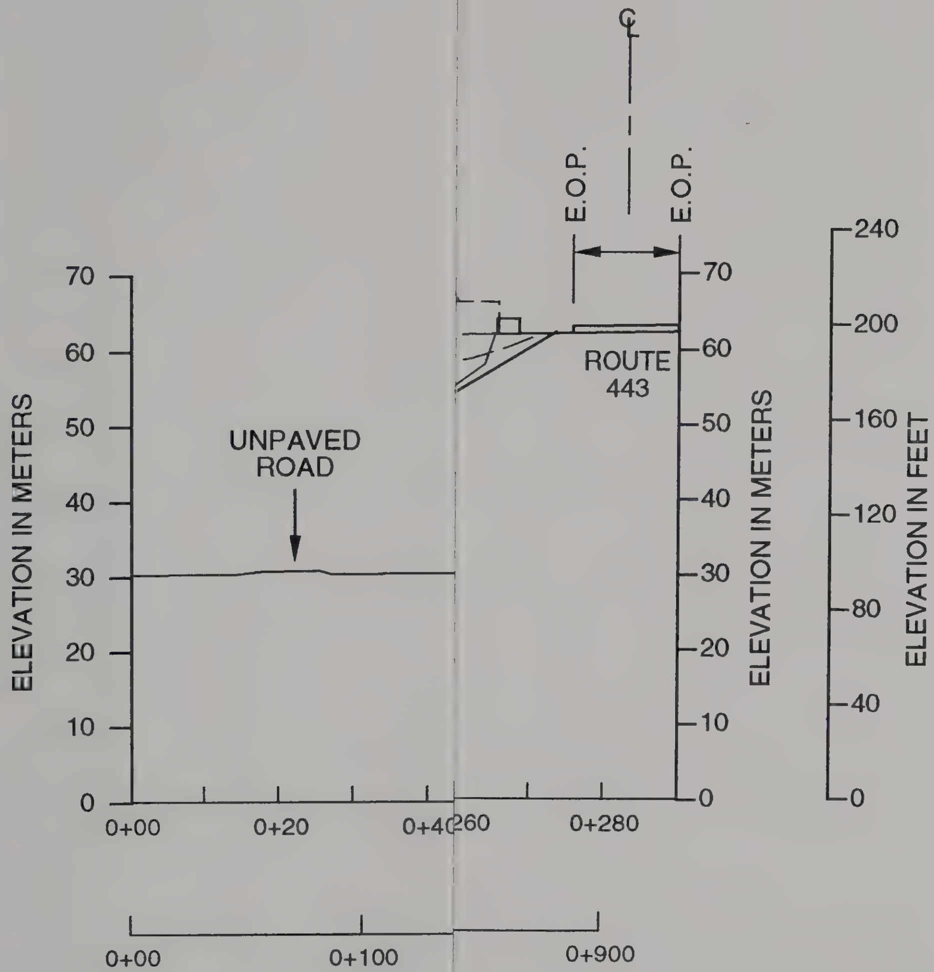
Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE LANDSLIDE SITE PLAN	DATE JUN 2000 JOB NO. 1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 2

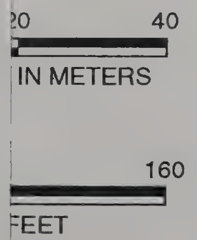
NOTE: NYSDOT PHOTO, MAY 19, 2000 LOOKING TOWARD THE NORTHEAST

LABELED FEATURES ARE APPROXIMATELY LOCATED

1280\FIGURE03 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE OBLIQUE PHOTOGRAPH	DATE JUN 2000 JOB NO. 1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 3



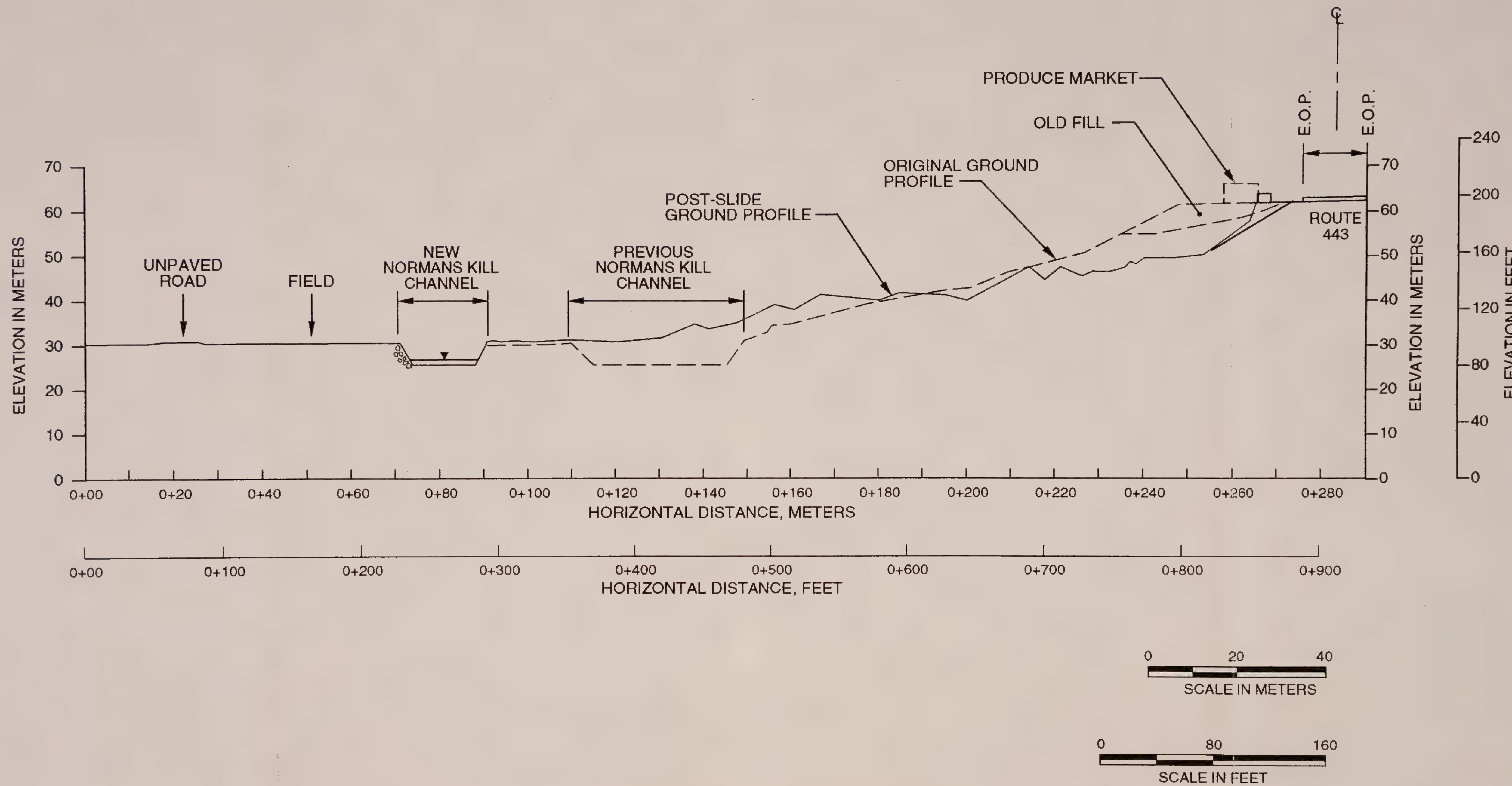


NOTE: ORIGINAL GROUND PR
1967 TOPOGRAPHY MA

POST-SLIDE CROSS SE
NYSDOT SURVEY, MAY

1280\FIGURE04 LJW

CROSS SECTION	DATE JUN 2000
	JOB NO. 1280
R 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 4

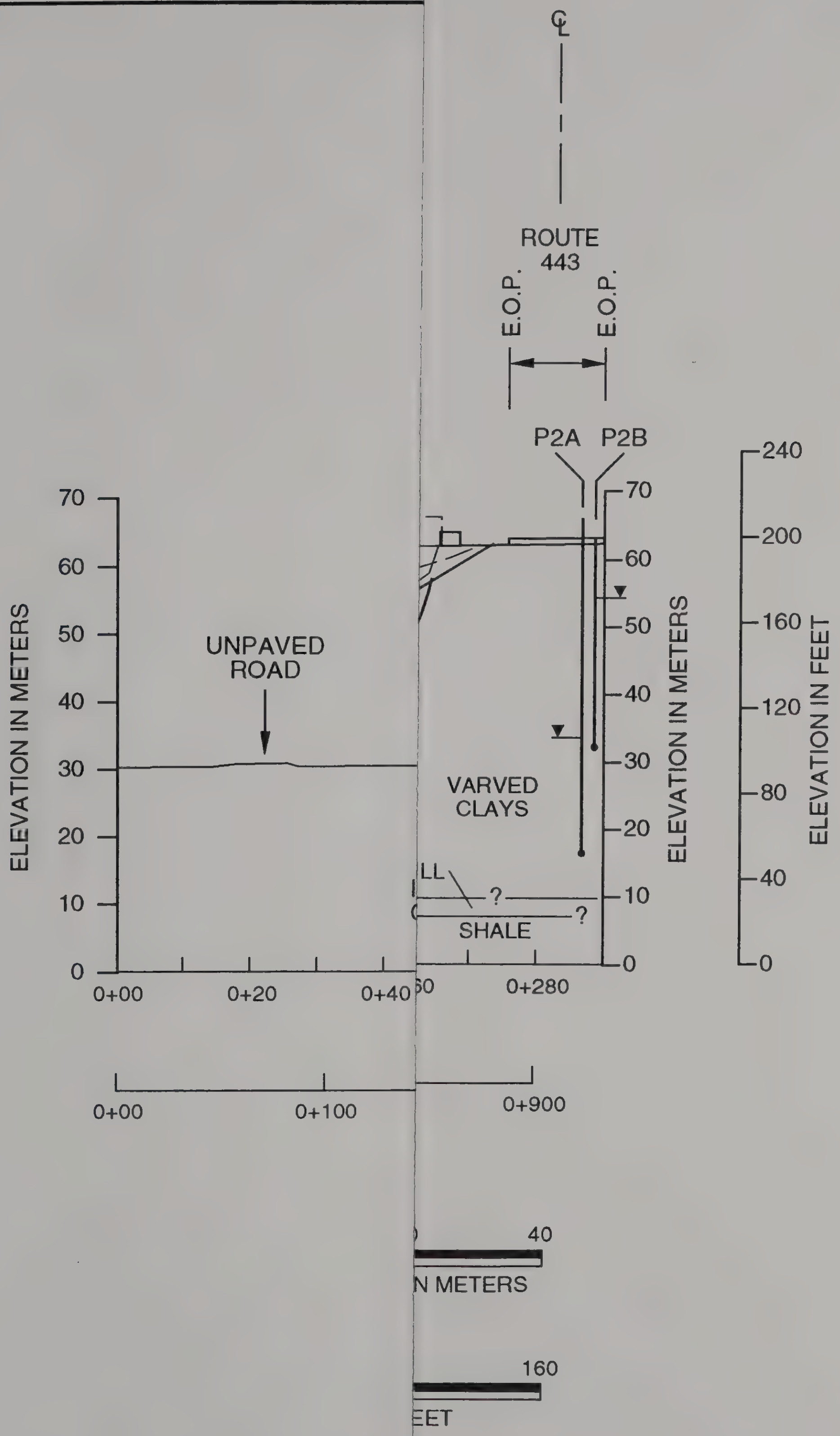


NOTE: ORIGINAL GROUND PROFILE BASED ON 1967 TOPOGRAPHY MAP

POST-SLIDE CROSS SECTION BASED ON NYSDOT SURVEY, MAY 24, 2000

1280\FIGURE04 LJW

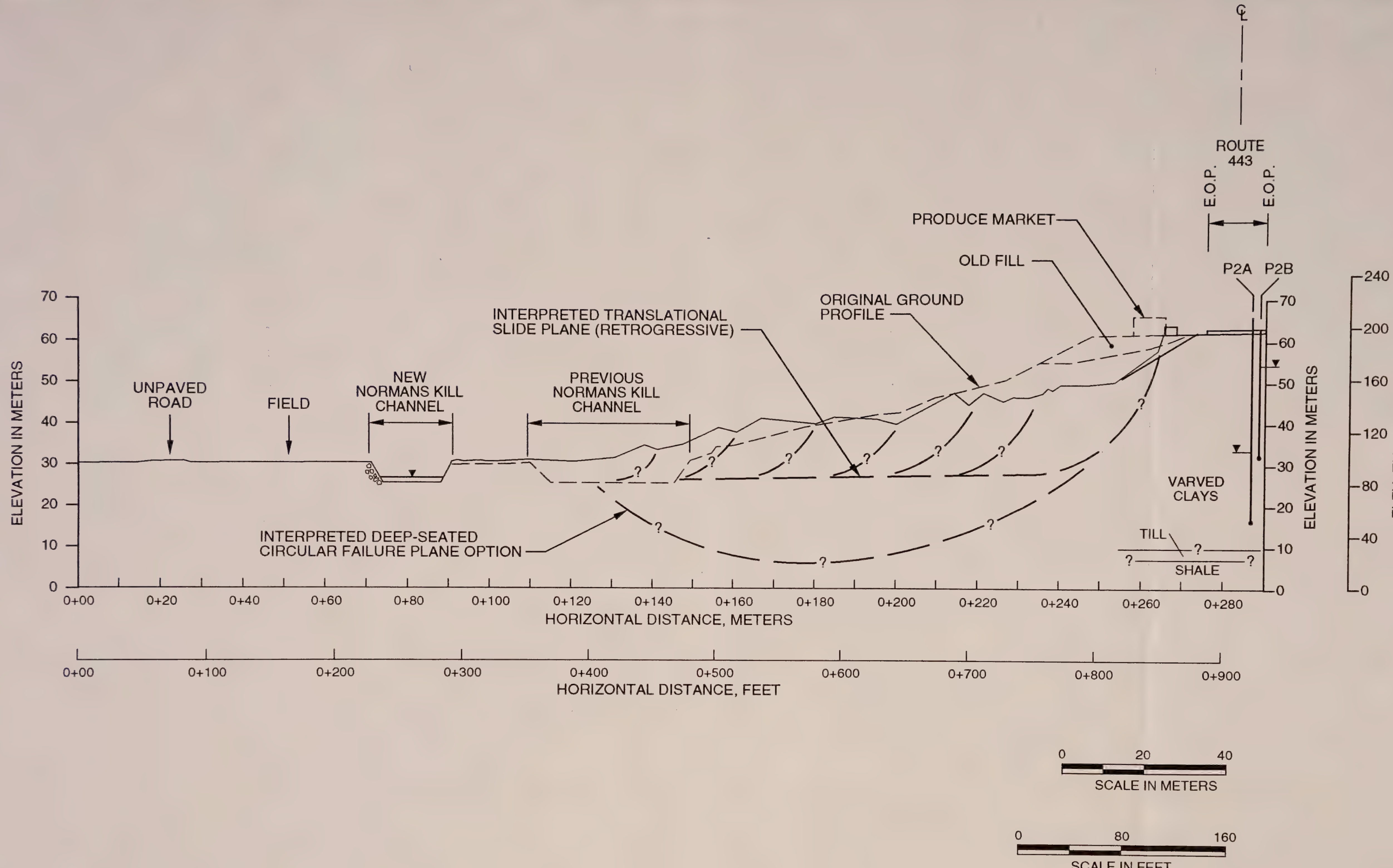
Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE CROSS SECTION	DATE JUN 2000
		JOB NO. 1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 4



NOTE: GROUNDWATER LEVEL
MEASURED ON 6-2-00

1280\FIGURE05 LJW

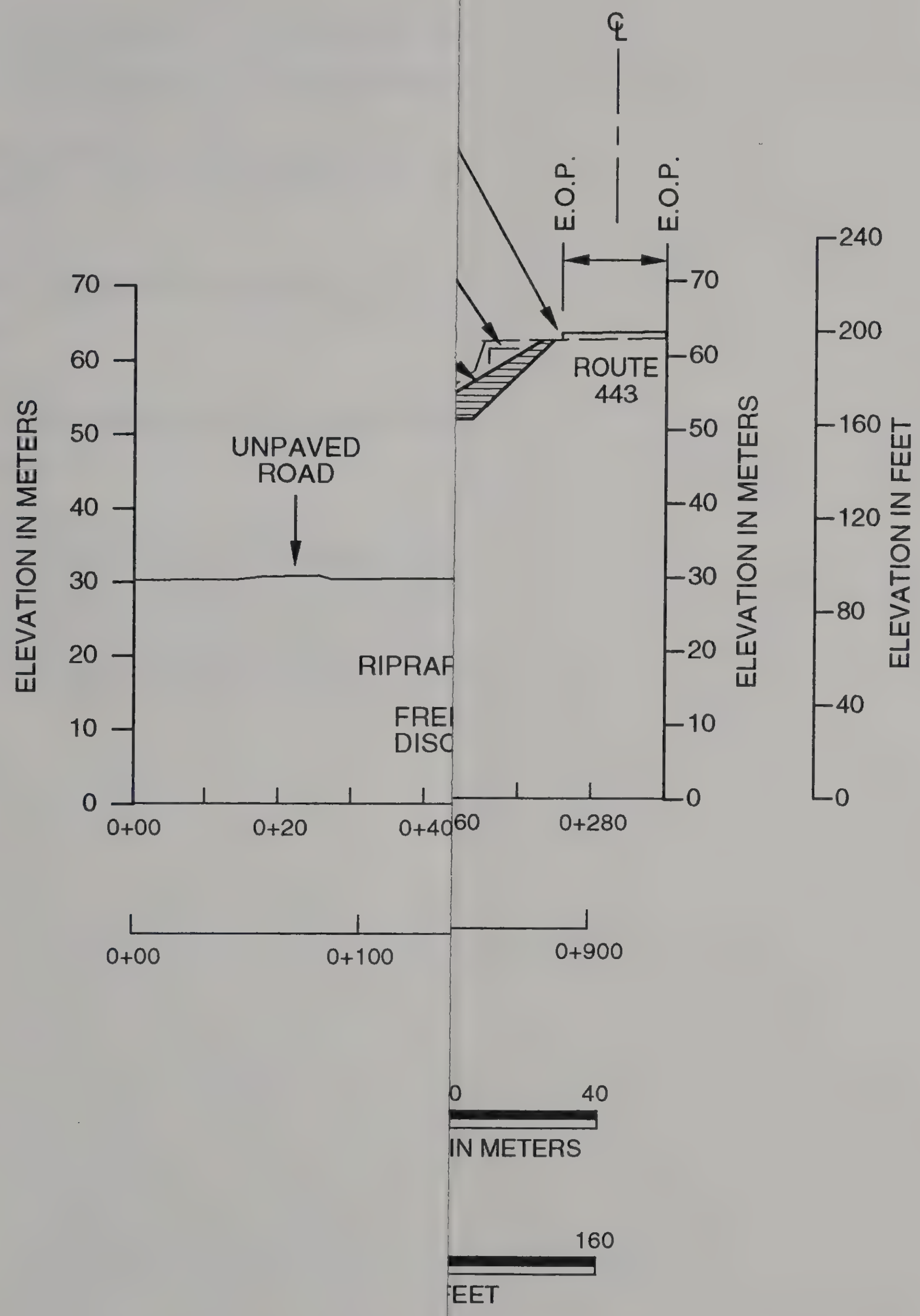
TERPRETED SUBSURFACE CROSS SECTION	DATE JUN 2000
	JOB NO. 1280
443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 5



NOTE: GROUNDWATER LEVELS IN P2A AND P2B
MEASURED ON 6-2-00

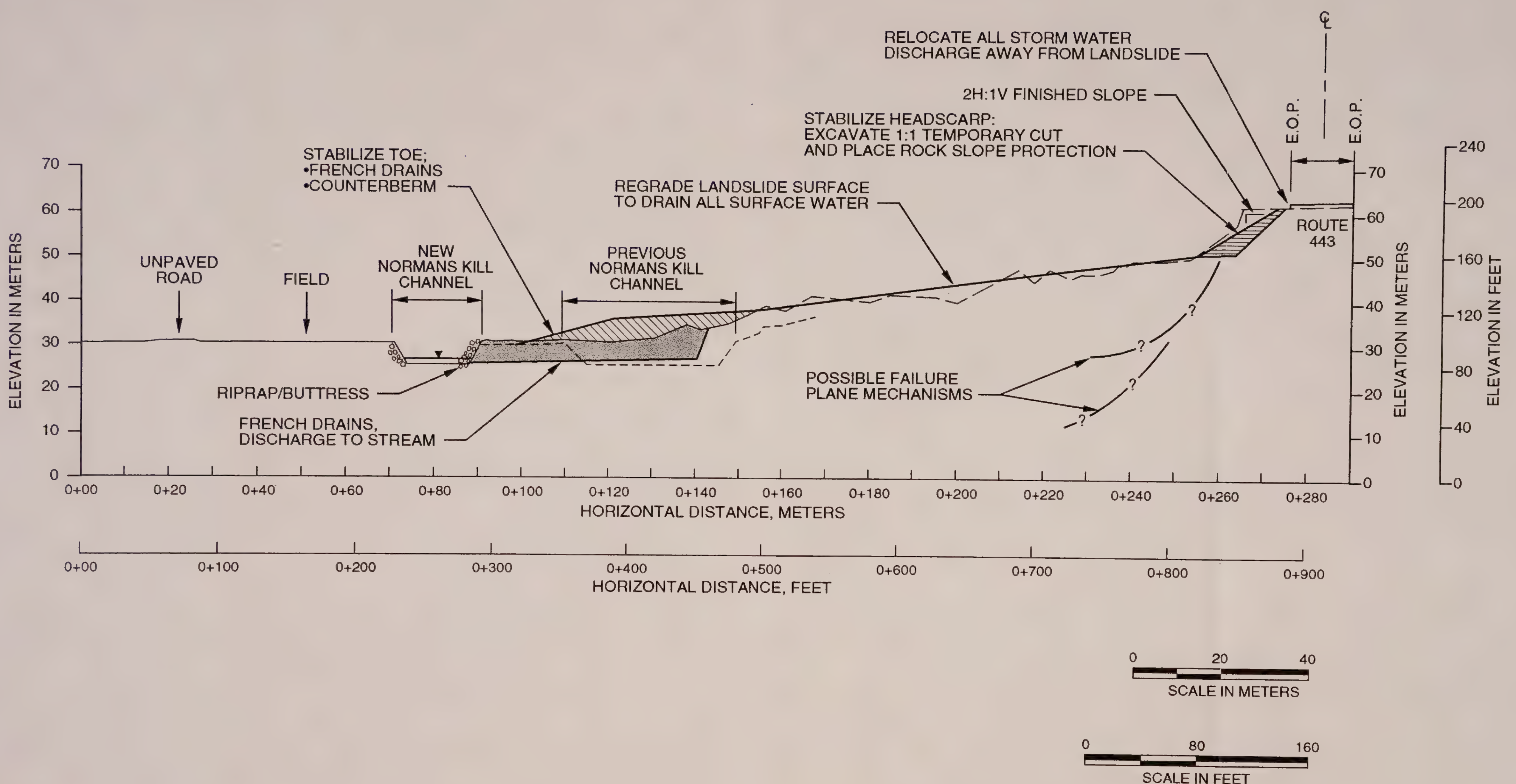
1280\FIGURE05 LJW

Landslide Technology 10250 S.W. Greenburg Rd. Portland, OR 97223	TITLE INTERPRETED SUBSURFACE CROSS SECTION	DATE JUN 2000
		JOB NO. 1280
	JOB SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 5



1280\FIGURE06 LJW

MITIGATION CONCEPTS CROSS SECTION	DATE JUN 2000
443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	JOB NO. 1280
	FIG. 6



1280\FIGURE06 LJW

Landslide Technology
10250 S.W. Greenburg Rd.
Portland, OR 97223

TITLE: MITIGATION CONCEPTS CROSS SECTION	DATE: JUN 2000
	JOB NO. 1280
JOB: SR 443 DELAWARE AVE. LANDSLIDE ELSMERE, NEW YORK	FIG. 6

APPENDIX A: EXPLORATIONS, INSTRUMENTATION TESTING

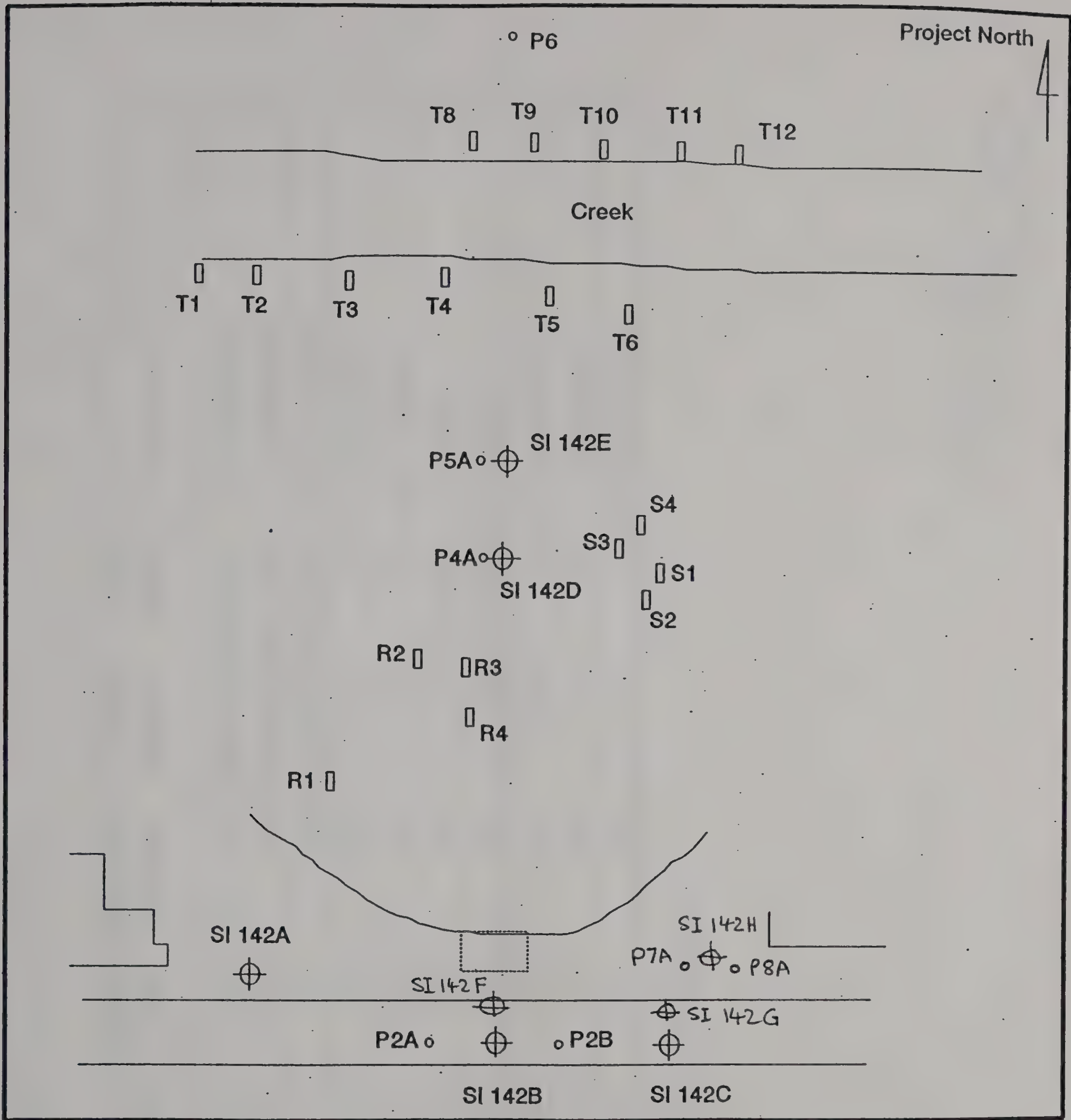
The locations of subsurface exploration borings and survey prisms are schematically illustrated on the following page.

The inclinometers installed to date, above the headscarp, are indicating 'no movement'.

The observation well data and installation information are included in this appendix.

Survey prism data is included herein for the locations where active movement as been recorded.

The results of the first phase of laboratory testing is also included. The results are a preliminary modification of the soil properties; however, additional testing has been programmed to check and refine the parameters. Engineering judgment will be necessary to select parameters for analysis and design.



Slope Indicator

Piezometer

Survey Prism

Not to Scale

Movement Orientation + −

Instrumention Layout

Prepared by: John C. Iori
Geotechnical Instrumention Bureau
June 1, 2000

Delaware Avenue Slide
Down–hole Instrument Summary

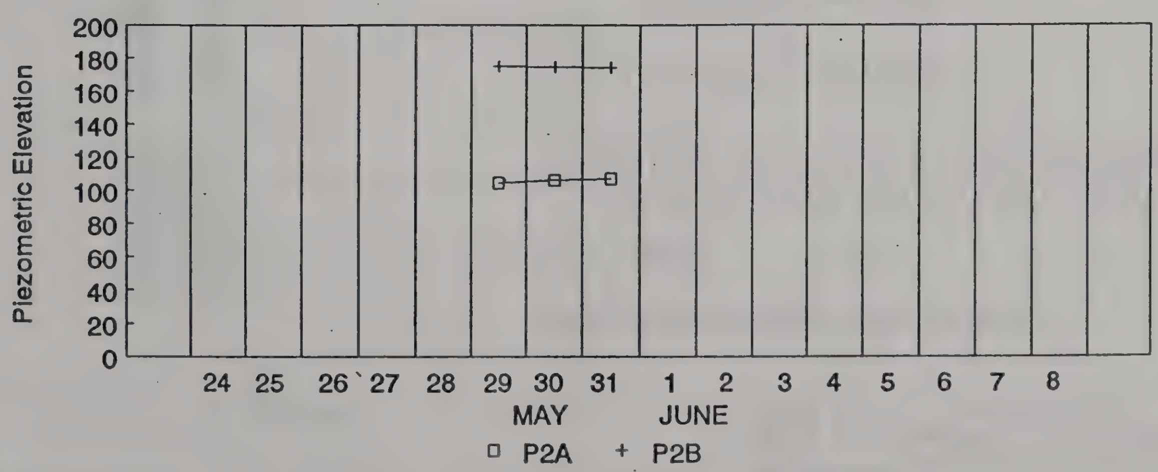
Instrument	Borehole Designation	OGS	Depth* (feet)	Instrument Elevation	Initial Reading	Latest Reading	Summary
SI 142A	NA	NA	88	NA	05/22/2000	05/31/2000	No movement.
SI 142B	NA	NA	120	NA	05/25/2000	05/31/2000	No movement.
SI 142C	NA	NA	130	NA	05/28/2000	05/31/2000	No movement.
SI 142D	NA	NA	NA	NA	NA	NA	Not Installed.
SI 142E	NA	NA	NA	NA	NA	NA	Not Installed.
P2A[1]	NA	NA	150	NA	NA	05/31/2000	See Plot.
P2B[1]	NA	NA	80	NA	NA	05/31/2000	See Plot.
P4A[2]	NA	NA	NA	60	NA	NA	Not Installed.
P5A[2]	NA	NA	NA	60	NA	NA	Not Installed.
P6[1]	NA	NA	NA	NA	NA	NA	Not Installed.

* Depth of Instrument. Bottom of hole may be deeper.
[1] Open Well Piezometer.
[2] Vibrating Wire Piezometer.

Prepared by: John C. Iori
Geotechnical Engineering Bureau
01–Jun–2000

TABLE OF PIEZOMETRIC READINGS
DELAWARE AVENUE
PIN 1090.13

HOLE:	NA		NA		NA	
PSN/BORNUM:	NA		NA		NA	
PIEZOMETER:	P2A		P2B		P6	
STATION/OFFSET:	NA		NA		NA	
SURF. ELEV.:	202.72		202.72		100.00 Estimate	
BOTTOM DEPTH:	150		80		NA	
COMPLETED:	May 28, 2000		May 27, 2000		NA	
DATE	Depth to Water (Feet)	Piezometric Elevation	Depth to Water (Feet)	Piezometric Elevation	Depth to Water (Feet)	Piezometric Elevation
29–May–2000	98.4	104.3	28.0	174.7		
30–May–2000	96.7	106.0	28.4	174.4		
31–May–2000	95.9	106.8	28.5	174.3		
01–Jun–2000						



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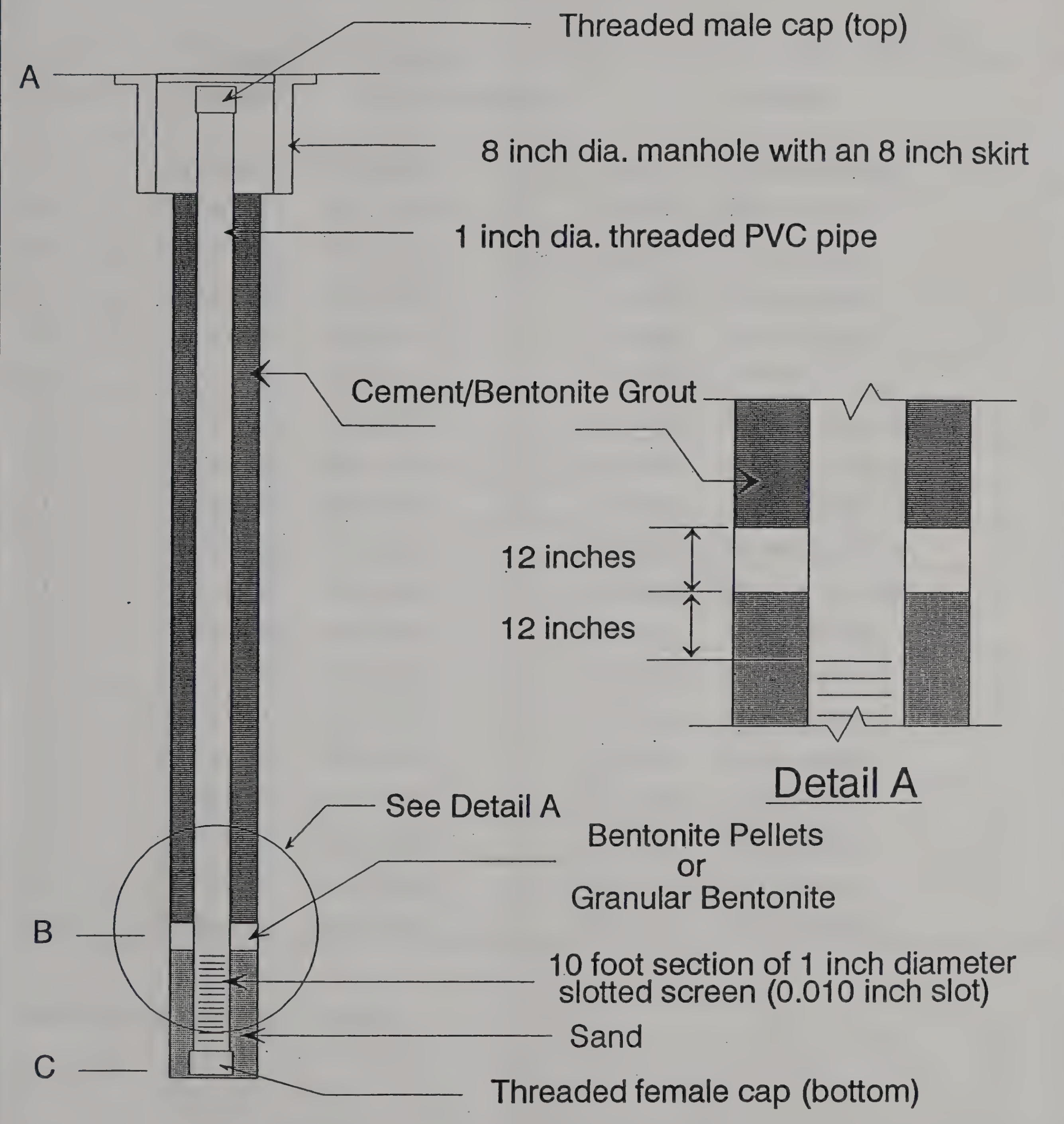
01–Jun–2000



ELEVATIONS:

	P2A	P2B
A	203	203
B	63	133
C	53	123

APPROVED 19

DIRECTOR
GEOTECHNICAL ENGINEERING BUREAU

REGION NO. MO
COUNTY N/A
DWG. NO. MO 1903

STATE OF NEW YORK
DEPARTMENT OF TRANSPORTATION
TECHNICAL SERVICES DIVISION

TYPICAL OPEN
WELL PIEZOMETER
INSTALLATION DETAILS

Delaware Avenue Slide
Prism Survey Summary

Stake	Initial Reading	Latest Reading	Location	Summary
R1	05/23/2000	05/31/2000	See Plan.	No movement.
R2	05/23/2000	05/31/2000	See Plan.	No movement.
R3	05/23/2000	05/31/2000	See Plan.	No movement.
R4	05/23/2000	05/31/2000	See Plan.	No movement.
S1	05/26/2000	06/01/2000	See Plan.	No movement.
S2	05/26/2000	06/01/2000	See Plan.	No movement.
S3	05/26/2000	06/01/2000	See Plan.	Moving. See plot.
S4	05/26/2000	06/01/2000	See Plan.	Moving. See plot.
T1	05/24/2000	05/31/2000	See Plan.	No movement.
T2	05/24/2000	05/25/2000	See Plan.	No longer visible.
T3	05/24/2000	05/31/2000	See Plan.	Moving. See plot.
T4	05/24/2000	05/31/2000	See Plan.	No movement.
T5	05/24/2000	05/31/2000	See Plan.	No movement.
T6	05/24/2000	05/31/2000	See Plan.	No movement.
T8	05/26/2000	05/31/2000	See Plan.	No movement.
T9	05/26/2000	05/31/2000	See Plan.	No movement.
T10	05/26/2000	05/31/2000	See Plan.	No movement.
T11	05/26/2000	05/31/2000	See Plan.	No movement.
T12	05/26/2000	05/31/2000	See Plan.	No movement.

Note: T7 is a temporary backsight.

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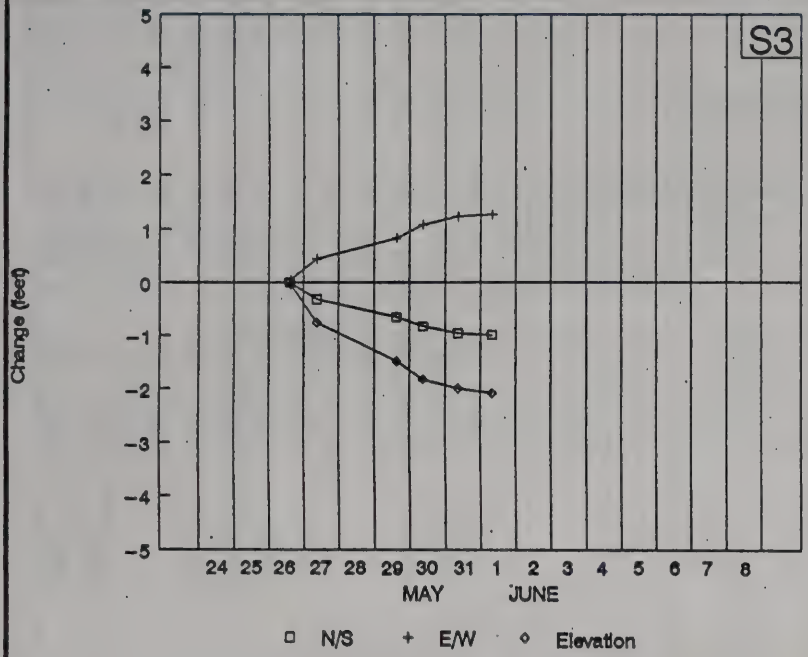
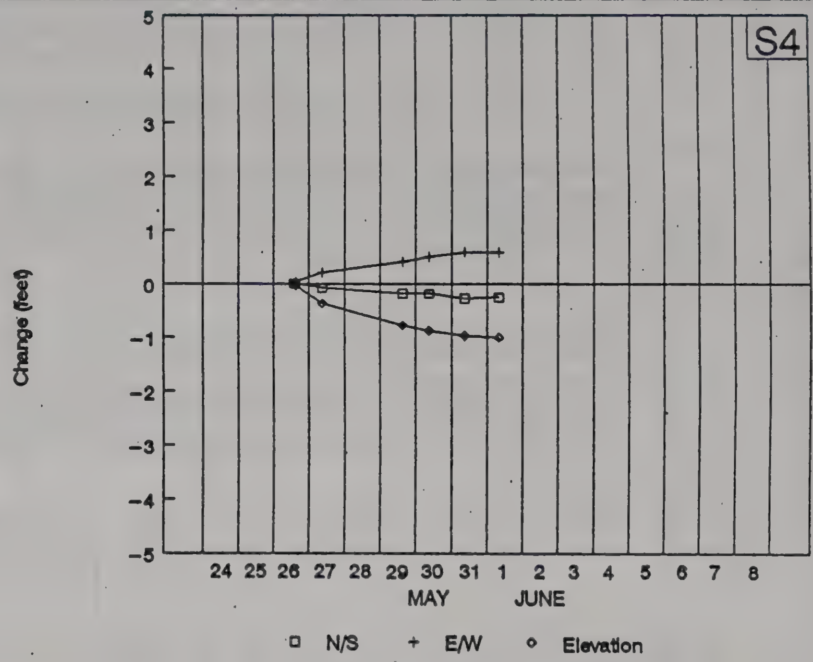
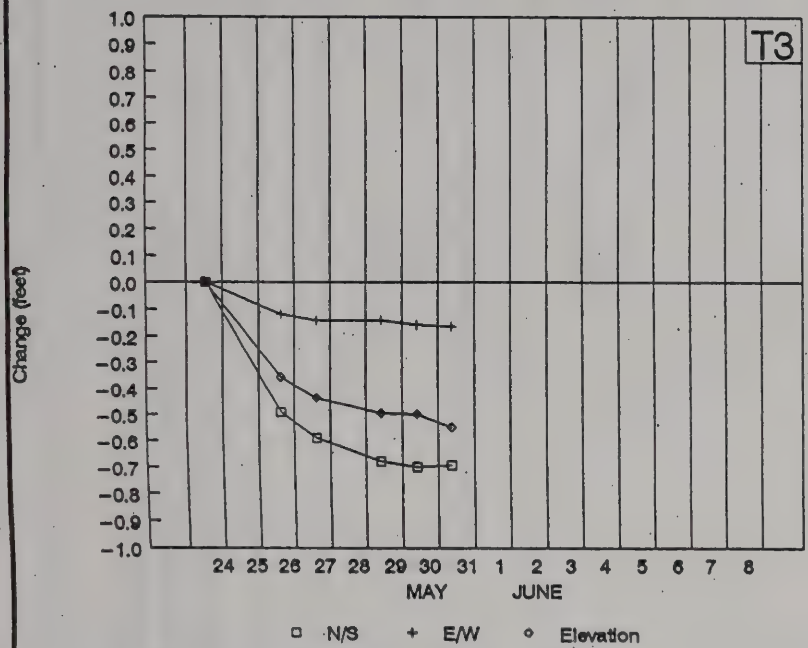






Delaware Avenue Slide
Prism Survey Summary
Prepared by: John C. Iori
Geotechnical Engineering Bureau

TABLE A1: SUMMARY RESULTS OF FIRST SERIES OF TESTS

Notes: 1. Borings FH-X-1, FH-X-2, and FH-X-3 are located at highway elevation.
2. Tests were performed by NYSDOT. Additional tests are being performed.

Boring, sample	Depth ft., (m)	Elev. ft., (m)	Natural Moisture Content	Liquid Limit	Plasticity Index	Effective CIU Triaxial	CU - total Stress Triaxial
FHX-3, J1	1, [0.3]	200, [61]	10%				
FHX-1, J2	5, [1.5]	194, [59.5]	33%				
FHX-2, J1	5, [1.5]	194, [59.5]	30%				
FHX-3, J2	10, [2.9]	191, [58.3]	25%				
FHX-1, J3	10, [2.9]	191, [58.3]	37%				
FHX-1, J3A	11, [3.2]	191, [58.0]	29%				
FHX-2, T2	10, [2.9]	191, [58.3]	39%				
FHX-1, J4	15, [4.5]	186, [56.7]	35%				
FHX-2, T3	15, [4.5]	186, [56.7]	40%				
FHX-1, J5	20, [6]	181, [55.2]	35%				
FHX-2, T4	20, [6]	181, [55.2]	29 - 38%	24%	6%		
FHX-3, J3	20, [6]	181, [55.2]	37%				
FHX-1, J6	25, [7.5]	176, [53.7]	31%				
FHX-2, T5	25, [7.5]	176, [53.7]	28%	25%	7%	phi=29	Su=207kPa @Pc=138kPa
FHX-2, T5	25, [7.5]	176, [53.7]				c=18kPa	Su=345kPa @Pc=276kPa
FHX-1, J7	30, [9]	171, [52.2]	35%				
FHX-3, J4	30, [9]	171, [52.2]	31%				
FHX-1, J8	35, [10.5]	166, [50.6]	34%				
FHX-2, T6	35, [10.5]	166, [50.6]	33%				
FHX-1, J9	40, [12]	161, [49.1]	36%				
FHX-3, J5	40, [12]	161, [49.1]	41%				
FHX-1, J10	45, [13.5]	156, [47.6]	42%				
FHX-2, T7	45, [13.5]	156, [47.6]					
FHX-1, J11	50, [15]	151, [46.1]	37%				
FHX-3, J6	50, [15]	151, [46.1]	38%				
FHX-1, J12	55, [16.6]	146, [44.5]	35%				
FHX-2, T8	55, [16.6]	146, [44.5]	38 - 45%	35%	15%	phi=18	
FHX-2, T8	55, [16.6]	146, [44.5]				c=10kPa	
FHX-2, T8	55, [16.6]	146, [44.5]					
FHX-1, J13	60, [18.1]	141, [43]	37%				
FHX-3, J7	60, [18.1]	141, [43]	35%				
FHX-1, J14	65, [19.7]	136, [41.5]	35%				
FHX-2, T9	65, [19.7]	136, [41.5]	35 -42%	34%	15%	phi=18	
FHX-2, T9	65, [19.7]	136, [41.5]				c=2kPa	
FHX-2, T9	65, [19.7]	136, [41.5]					
FHX-1, J15	70, [21.2]	131, [40]	36%				
FHX-3, J8	70, [21.2]	131, [40]	36%				
FHX-1, J16	75, [22.7]	126, [38.4]	36%				
FHX-2, T10	75, [22.7]	126, [38.4]	35 - 45%	34%	13%	phi=18	Su=209kPa @Pc=207kPa
FHX-2, T10	75, [22.7]	126, [38.4]				c=0kPa	Su=236kPa @Pc=310kPa
FHX-2, T10	75, [22.7]	126, [38.4]					Su=247kPa @Pc=414kPa
FHX-1, J17	80, [24.2]	121, [36.9]	36%				
FHX-3, J9	80, [24.2]	121, [36.9]	38%				

TABLE A1: SUMMARY RESULTS OF FIRST SERIES OF TESTS (Continued)

Notes:
1. Borings FH-X-1, FH-X-2, and FH-X-3 are located at highway elevation.
2. Tests were performed by NYSDOT. Additional tests are being performed.

Boring, sample	Depth ft., (m)	Elev. ft., (m)	Natural Moisture Content	Liquid Limit	Plasticity Index	Effective CIU Triaxial	CU - total Stress Triaxial
FHX-1, J18	85, [25.8]	116, [35.4]	40%				
FHX-2, T11	85, [25.8]	116, [35.4]	42 - 49%	40%	18%		
FHX-2, T11	85, [25.8]	116, [35.4]	42 - 49%	42%	21%	phi=17	Su=303kPa @Pc=469kPa
FHX-2, T11	85, [25.8]	116, [35.4]				c=25kPa	Su=311kPa @Pc=552kPa
FHX-2, T11	85, [25.8]	116, [35.4]					Su=334kPa @Pc=621kPa
FHX-1, J19	90, [27.3]	111, [33.9]	44%				
FHX-3, J10	90, [27.3]	111, [33.9]	33%				
FHX-2, T12	95, [28.9]	106, [32.3]	49%	35%	14%		Su=336kPa @Pc=517kPa
FHX-2, T12	95, [28.9]	106, [32.3]					Su=236kPa @Pc=621kPa
FHX-2, T12	95, [28.9]	106, [32.3]					Su=429kPa @Pc=724kPa
FHX-3, J11	100, [30.4]	101, [30.8]	42%				
FHX-1, J20	100, [30.4]	101, [30.8]	38%				
FHX-2, T13	105, [31.9]	96, [29.3]	39%	26%	8%	phi=17	Su=386kPa @Pc=345kPa
FHX-2, T13	105, [31.9]	96, [29.3]				c=50kPa	Su=428kPa @Pc=448kPa
FHX-2, T13	105, [31.9]	96, [29.3]					Su=783kPa @Pc=586kPa
FHX-1, J21	110, [33.4]	91, [27.8]	32%				
FHX-3, J12	110, [33.4]	91, [27.8]	31%				
FHX-1, J22	120, [36.5]	81, [24.7]	30%				
FHX-3, J13	120, [36.5]	81, [24.7]	33%				
FHX-1, J23	130, [39.5]	71, [21.7]	28%				
FHX-3, J14	130, [39.5]	71, [21.7]	30%				
FHX-1, J24	140, [42.5]	61, [18.6]	27%				
FHX-3, J15	140, [42.5]	61, [18.6]	39%				
FHX-1, J25	170, [51.7]	31, [9.5]	9%				

APPENDIX B: PRELIMINARY STABILITY ANALYSES

The following conditions were evaluated to assess slope behavior and to confirm use of reasonable parameters:

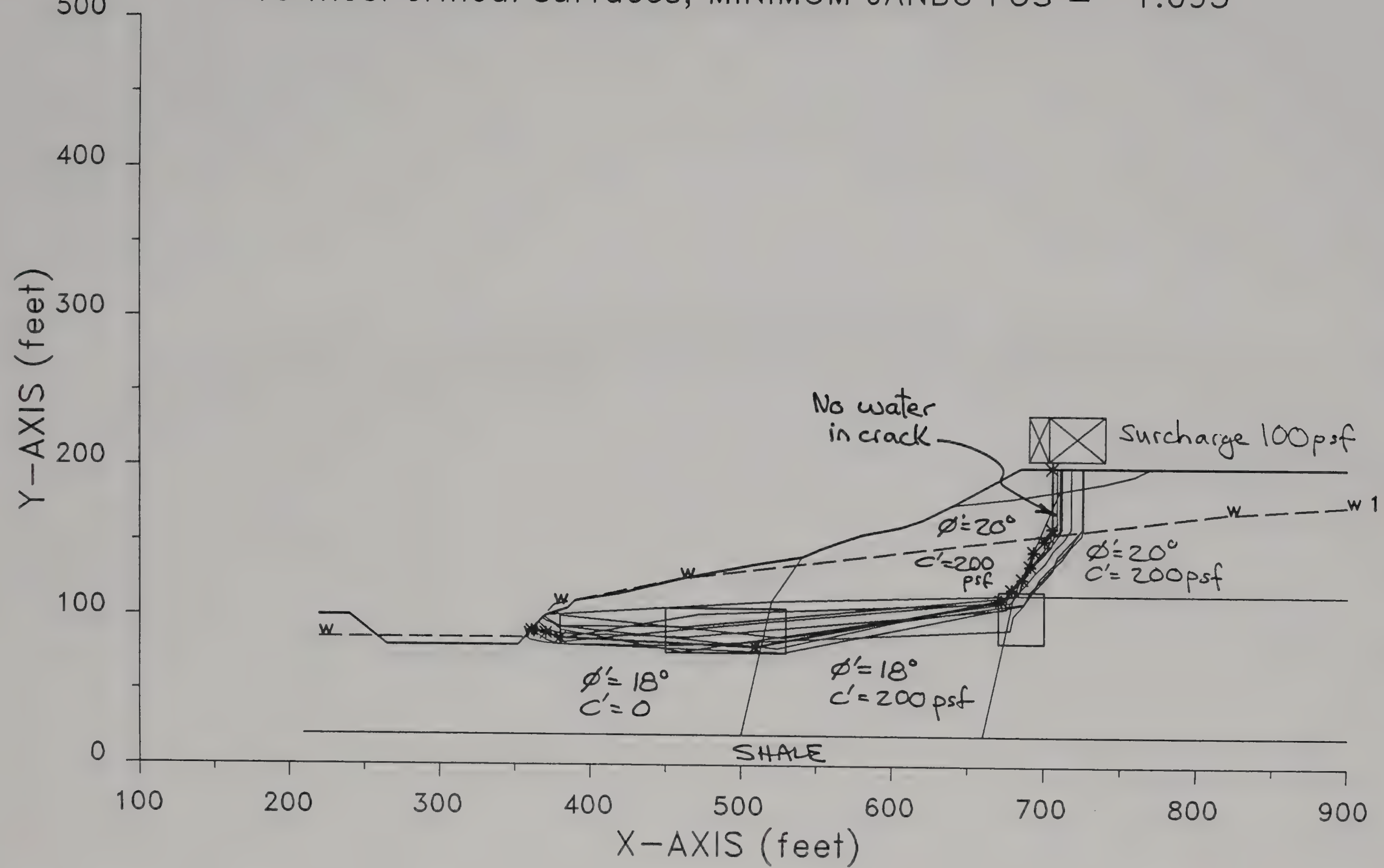
- Toe stability during elevated surface water and groundwater conditions,
- The impact of fill placement on slope stability,
- The initiation of the landslide,
- Stability achieved by the rapid slide movement and toe-building.

The stability analyses were performed using XSTABL software. Both total stress and effective stress analyses were used to model rapid, initial failure and longer-term partially drained conditions. The landslide topography was developed from existing topographic maps and a survey performed by NYSDOT the week following the start of the enlarged landslide. Groundwater conditions were based on groundwater seepage elevations and initial groundwater level measurements from the first two observation wells. Subsequent observation wells and piezometers should be used to refine groundwater assumptions. Shear strengths of the soils were initially estimated pending completion of laboratory testing. Several references were consulted to evaluate the reasonableness of strength estimates. Engineering judgment was used to select strength parameters for the preliminary and conceptual series of stability analyses.

The following analyses sheets show example cases that are representative for preliminary studies.

I_3CB 6-04-** 17:55

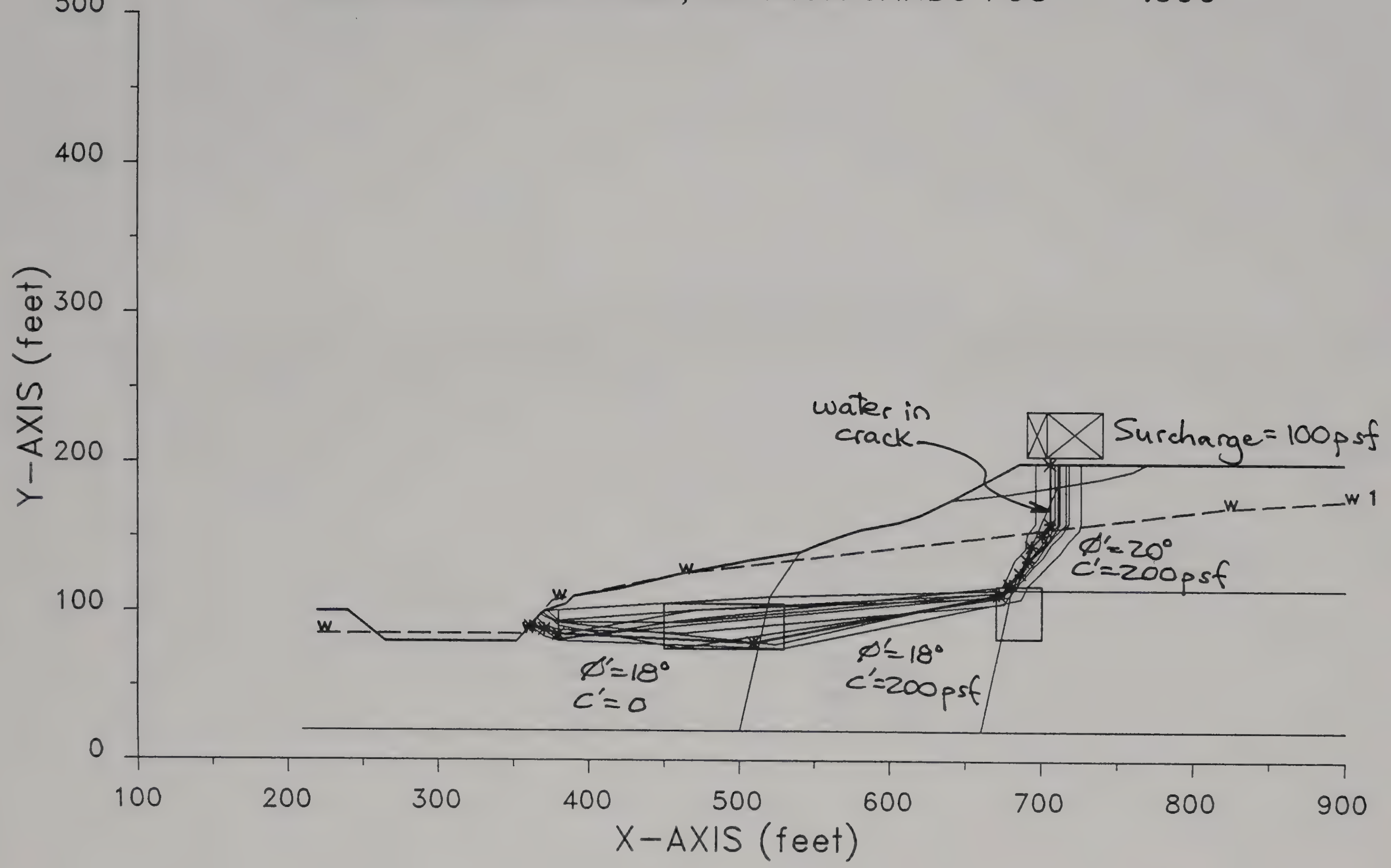
Normans Kill Slide, Initiate, surch
10 most critical surfaces, MINIMUM JANBU FOS = 1.093



I_3CC 6-04-** 17:56

Normans Kill Slide, Initiate, surch

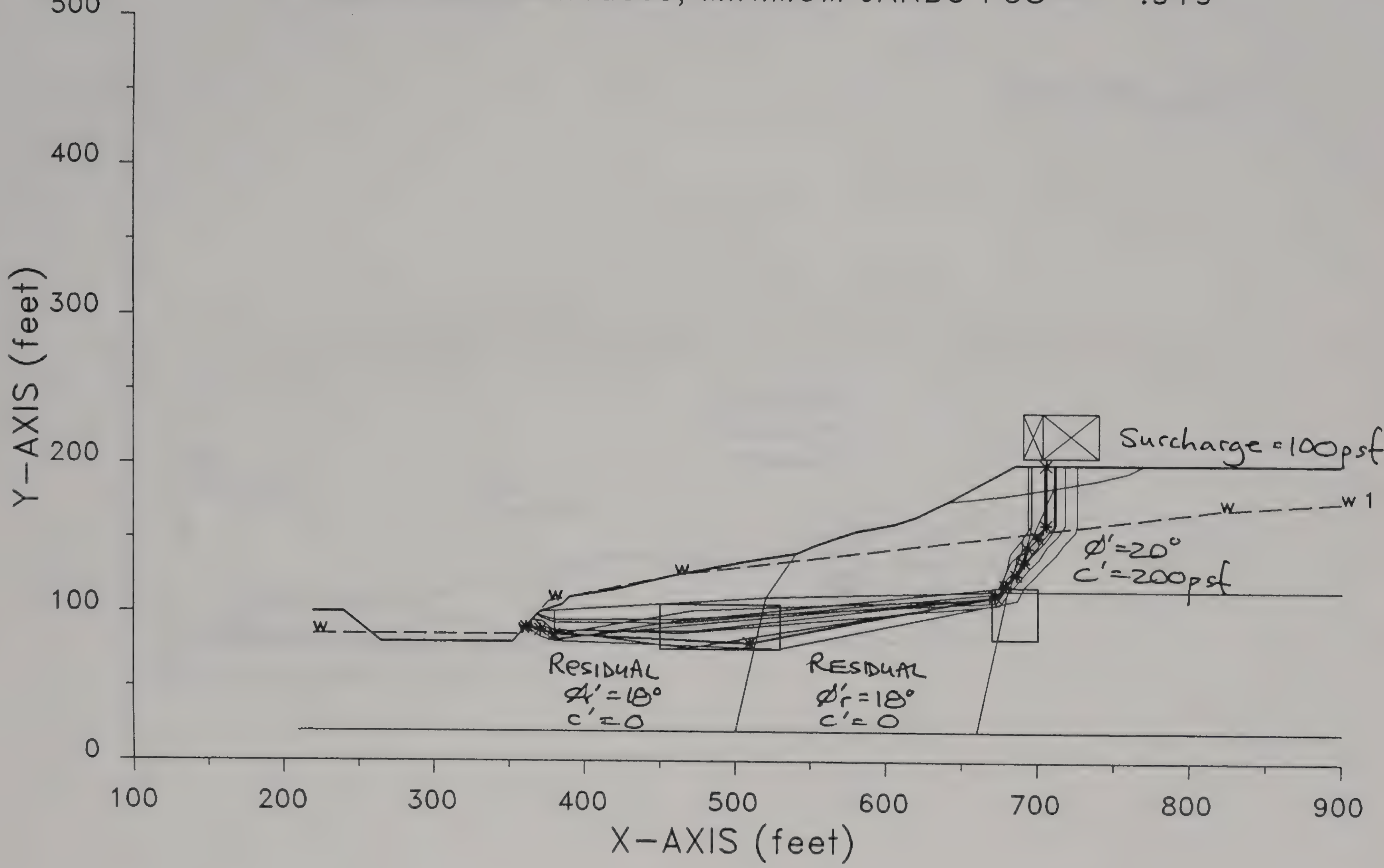
10 most critical surfaces, MINIMUM JANBU FOS = .990



I_3CD 6-04-** 18:01

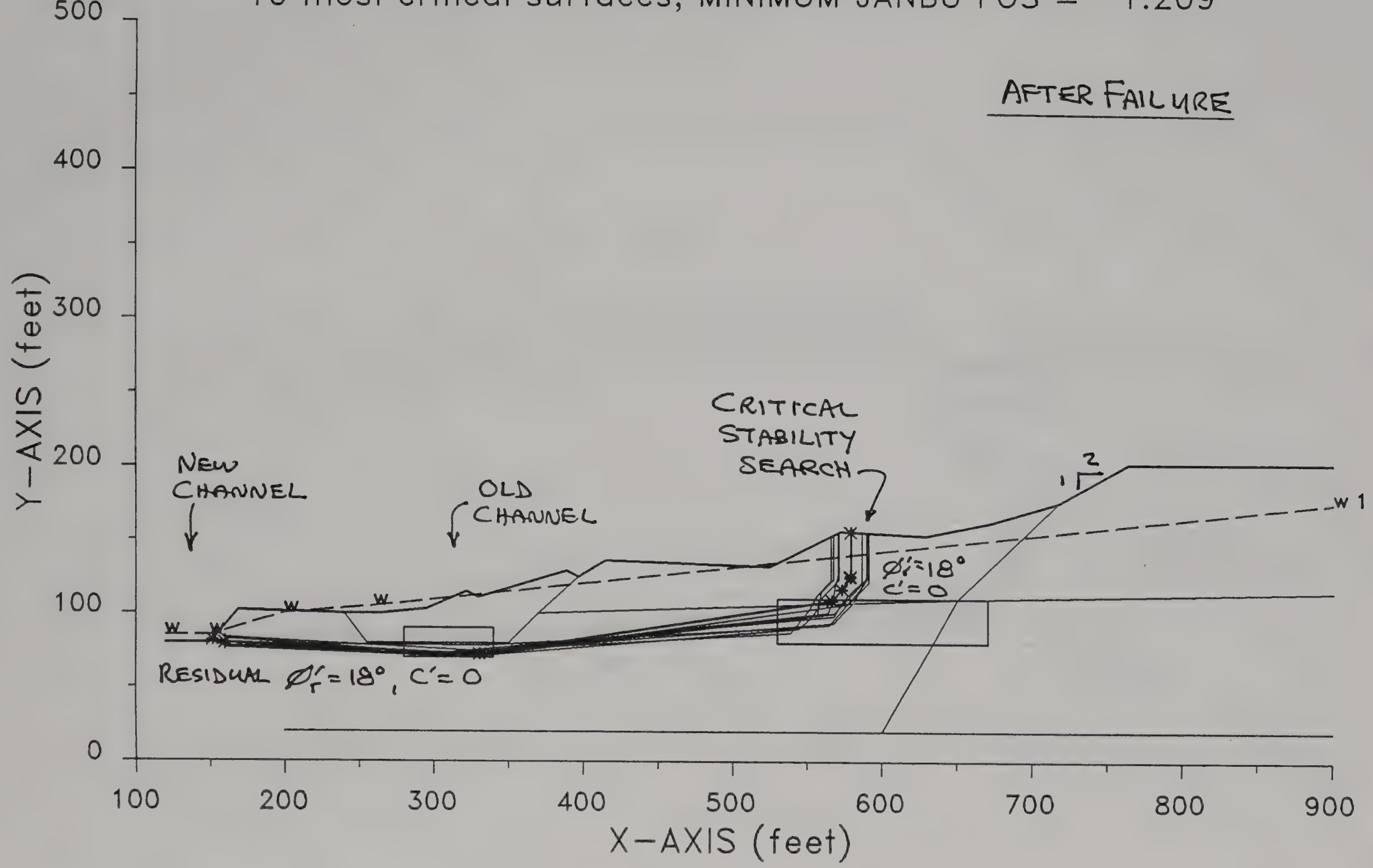
Normans Kill Slide,Initiate,Residual

10 most critical surfaces, MINIMUM JANBU FOS = .919



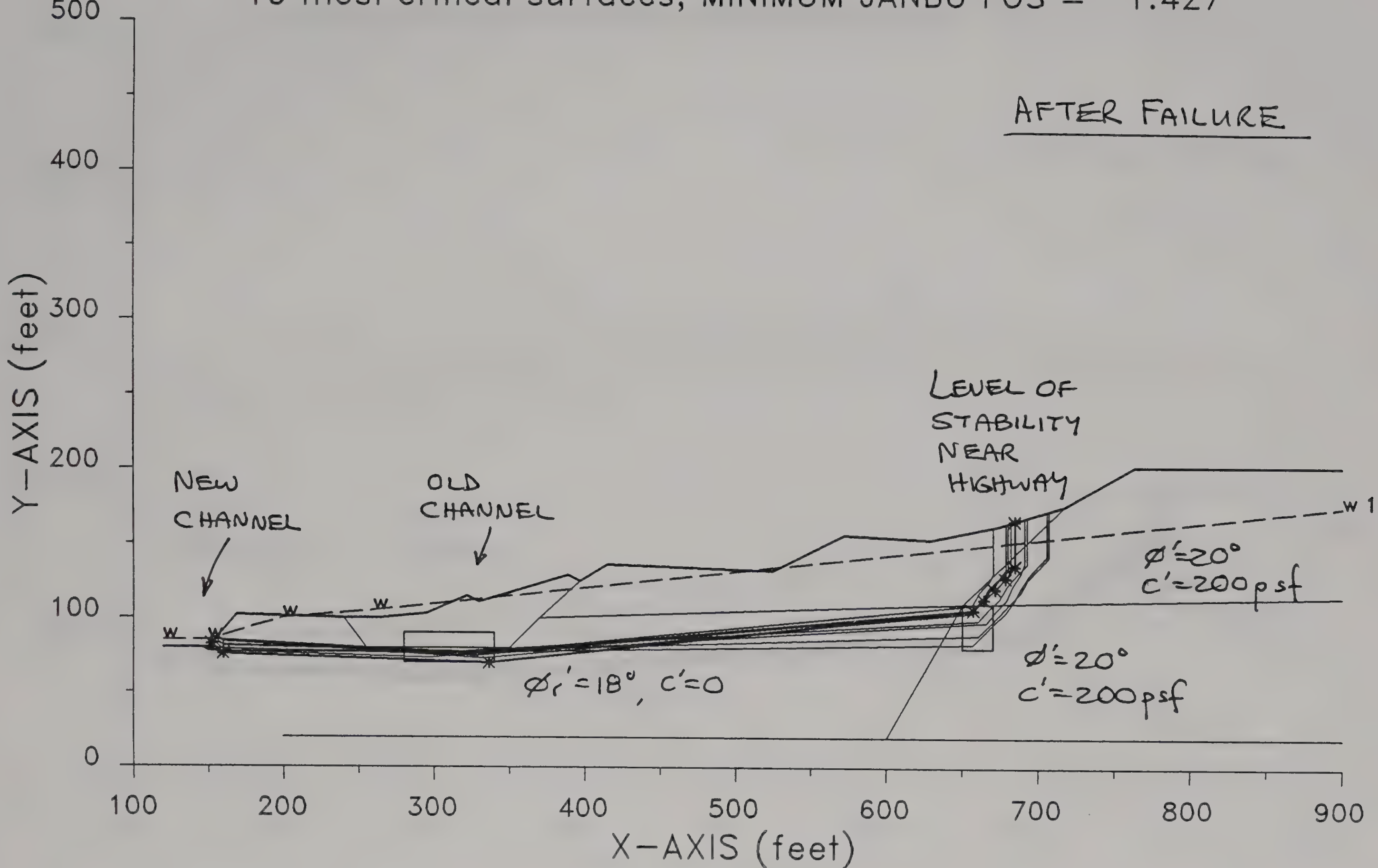
ST_1CA 6-04-** 18:24

Normans Kill, effectstress, gwt 175'
10 most critical surfaces, MINIMUM JANBU FOS = 1.209



ST_1CB 6-04-** 18:25

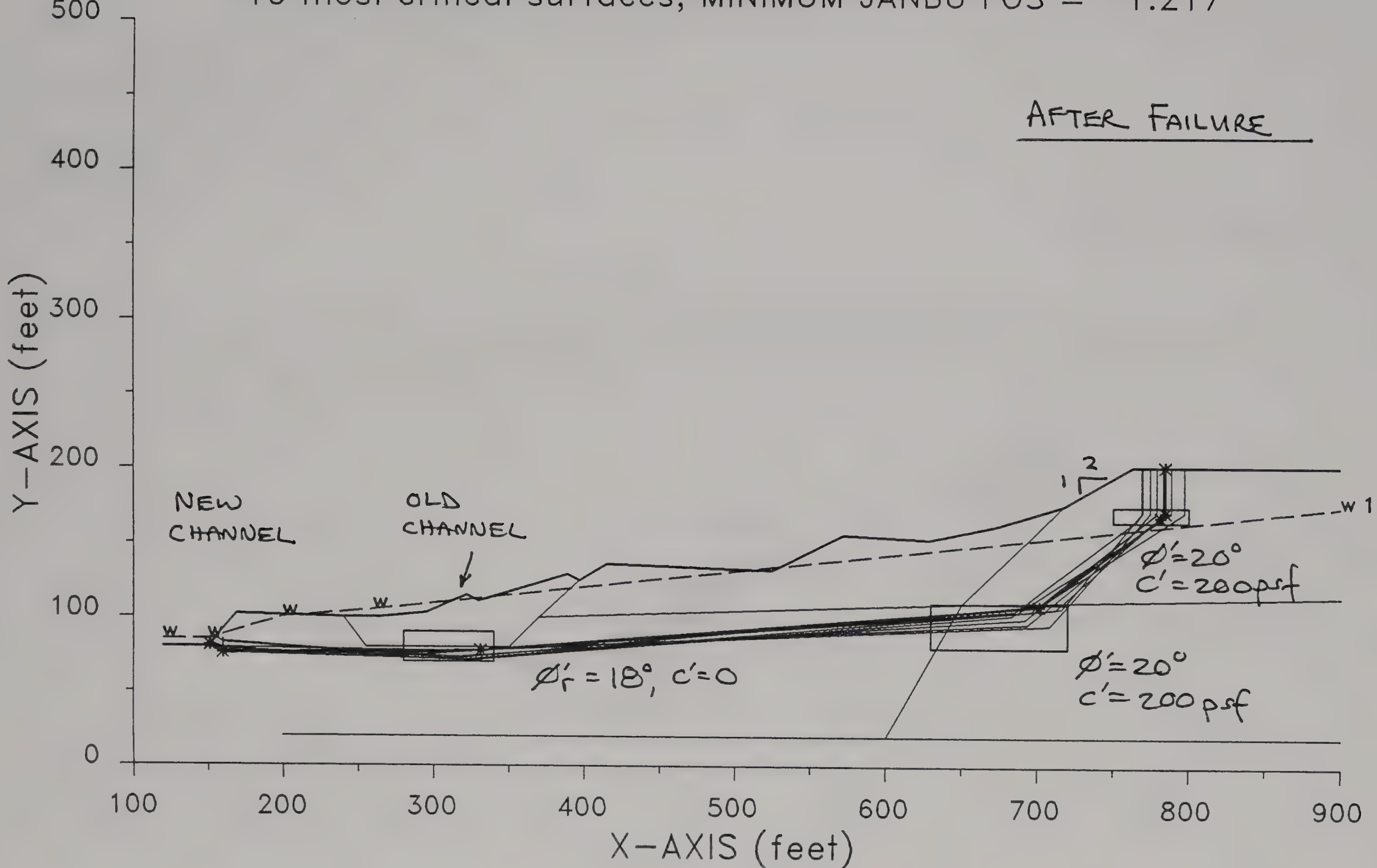
Normans Kill, effectstress, gwt 175'
10 most critical surfaces, MINIMUM JANBU FOS = 1.427



ST_1CC 6-04-** 18:28

Normans Kill, effectstress, gwt 175'

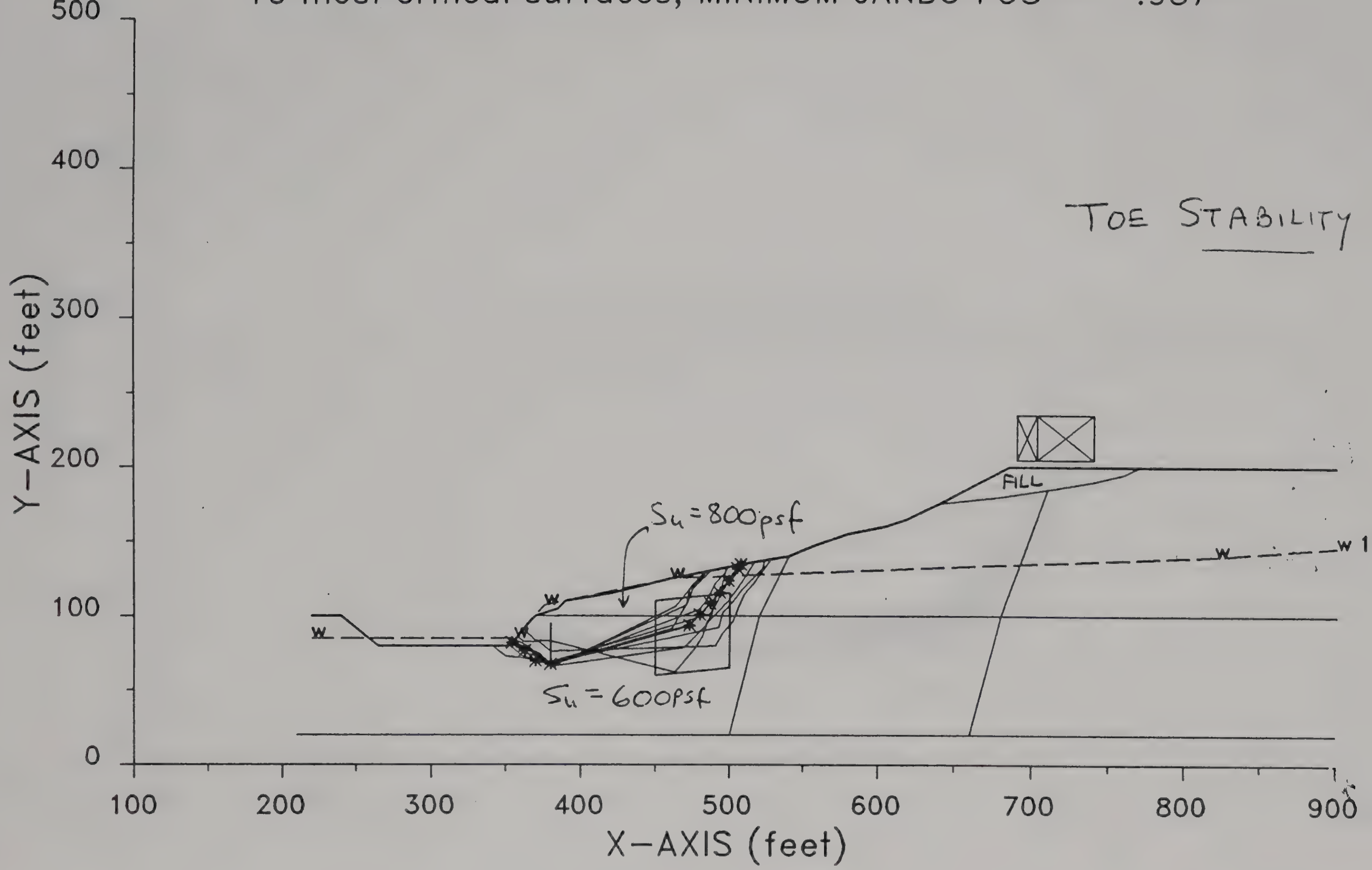
10 most critical surfaces, MINIMUM JANBU FOS = 1.217



I_3A 5-29-** 14:01

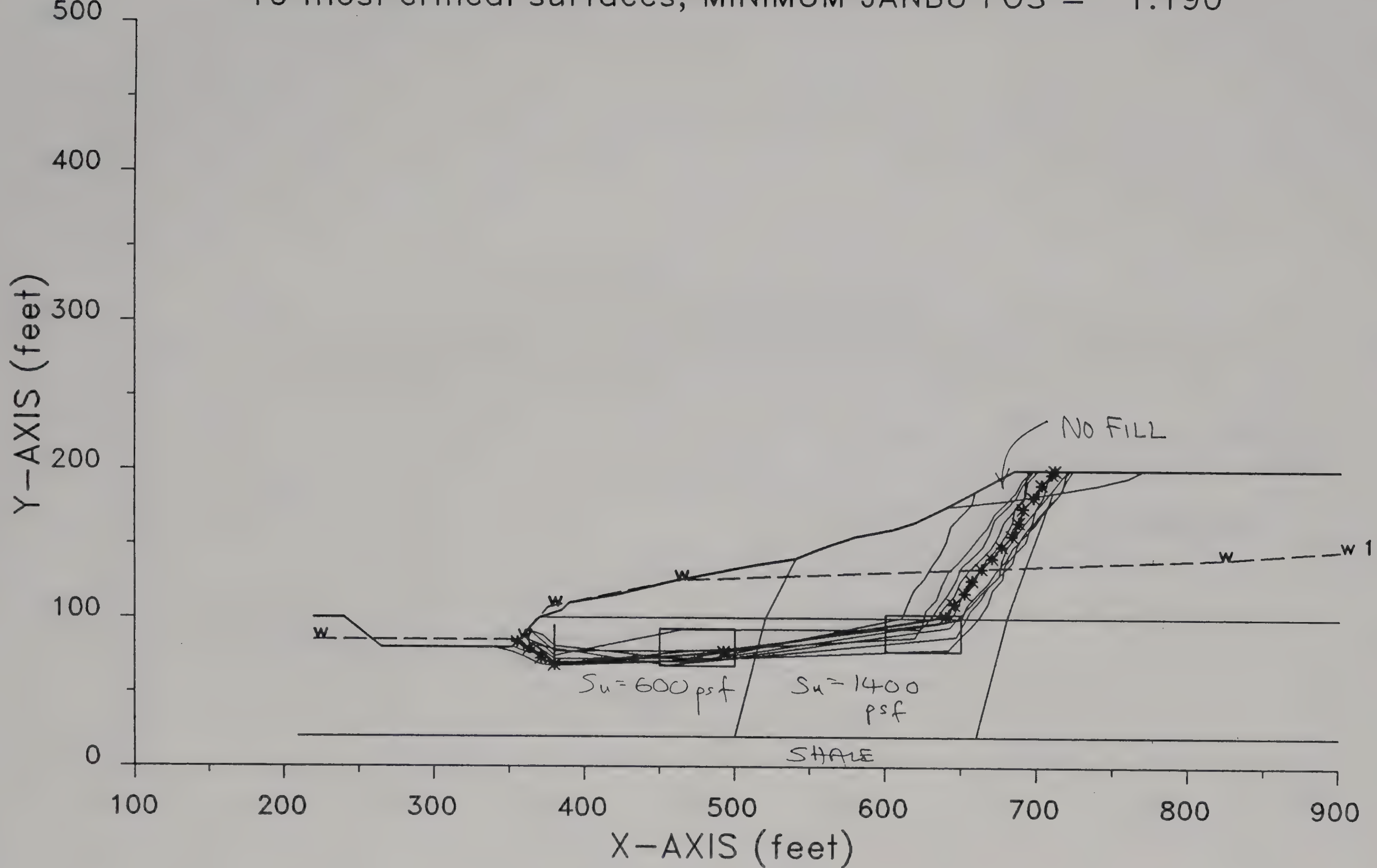
Normans Kill Slide, Initiate, toe

10 most critical surfaces, MINIMUM JANBU FOS = .957



I_3B1 5-30-** 16:05

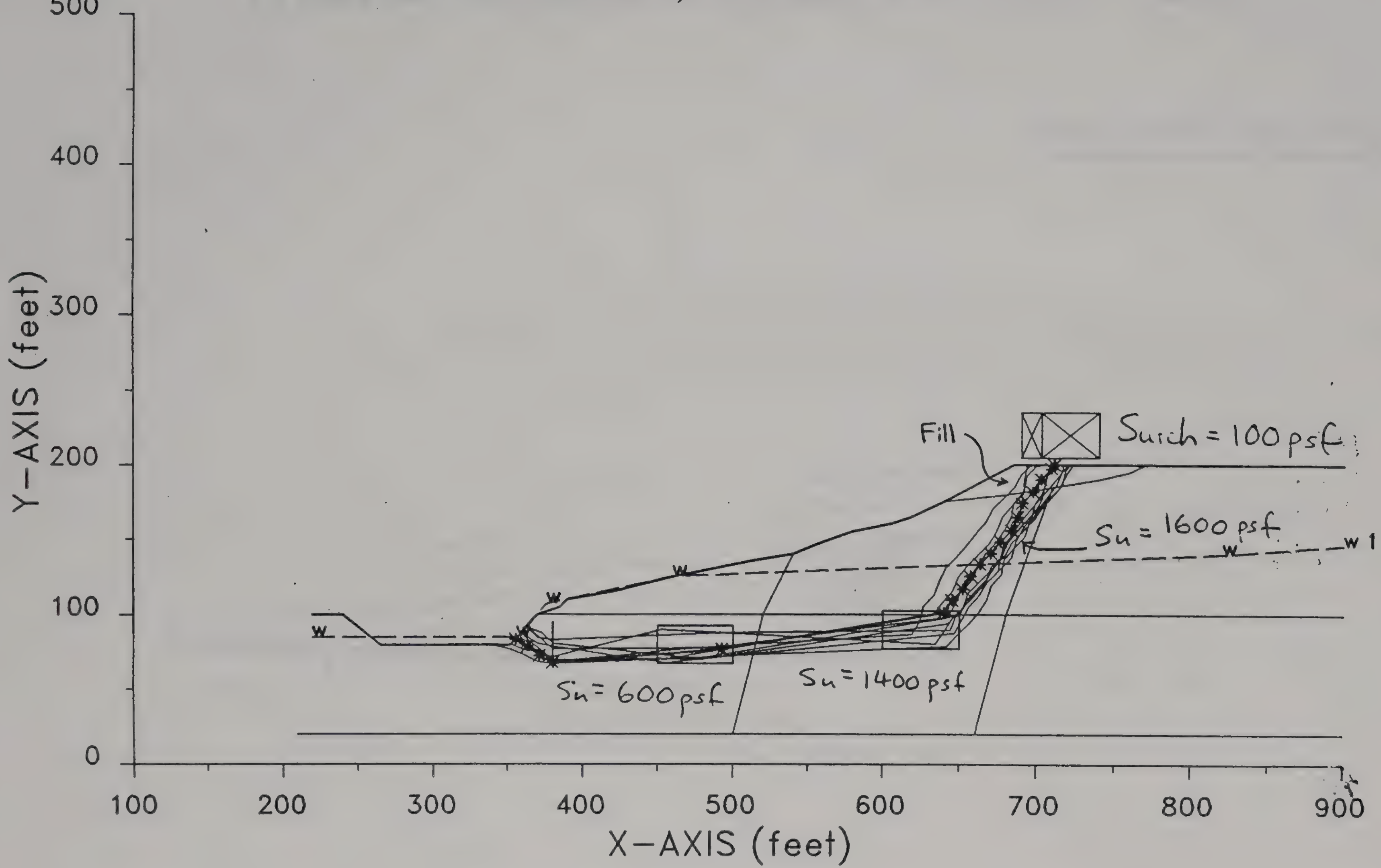
Normans Kill Slide, Initiate,no fill
10 most critical surfaces, MINIMUM JANBU FOS = 1.190

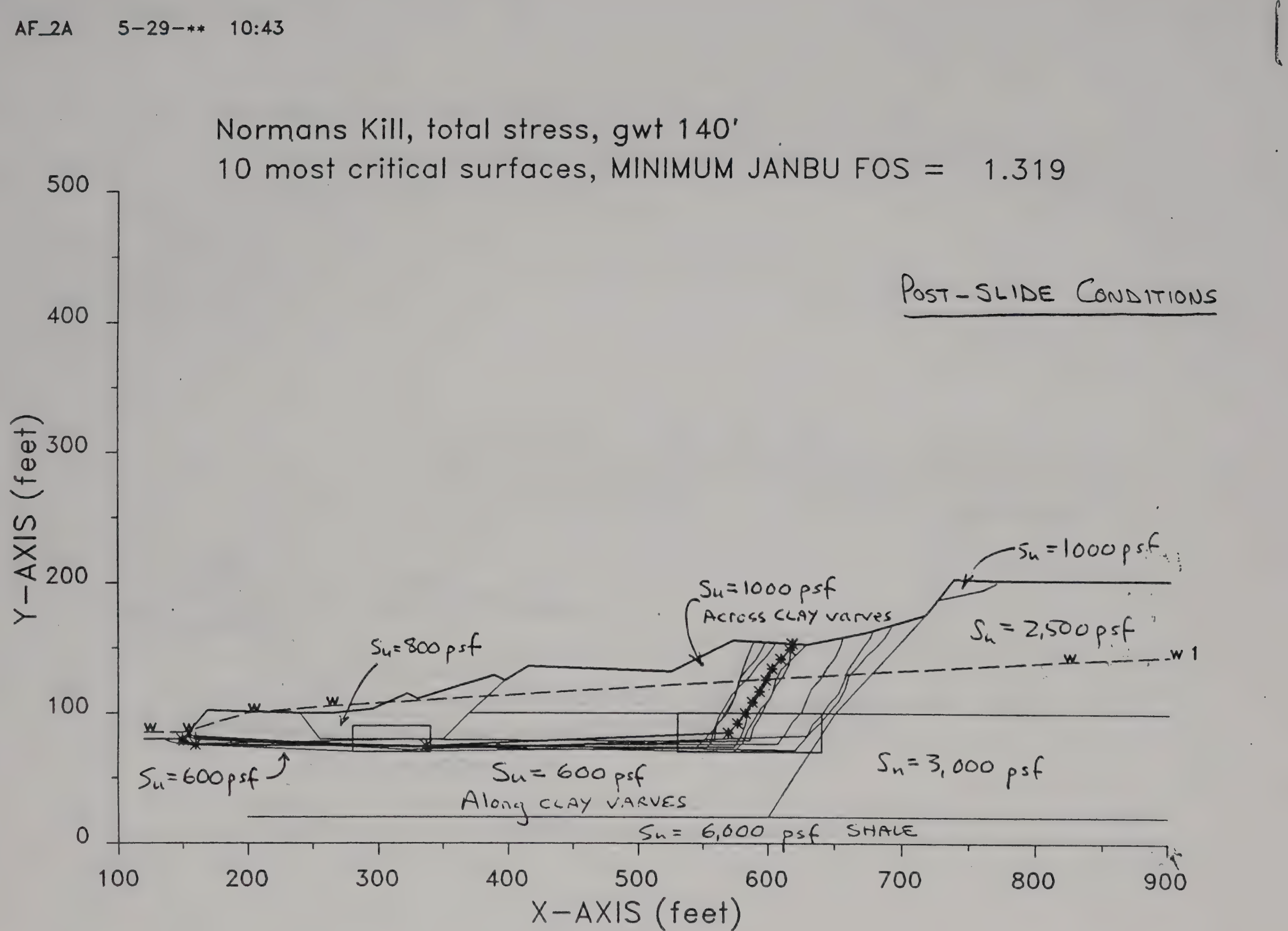


I_3B 5-29-** 14:12

Normans Kill Slide, Initiate, surch

10 most critical surfaces, MINIMUM JANBU FOS = 1.024

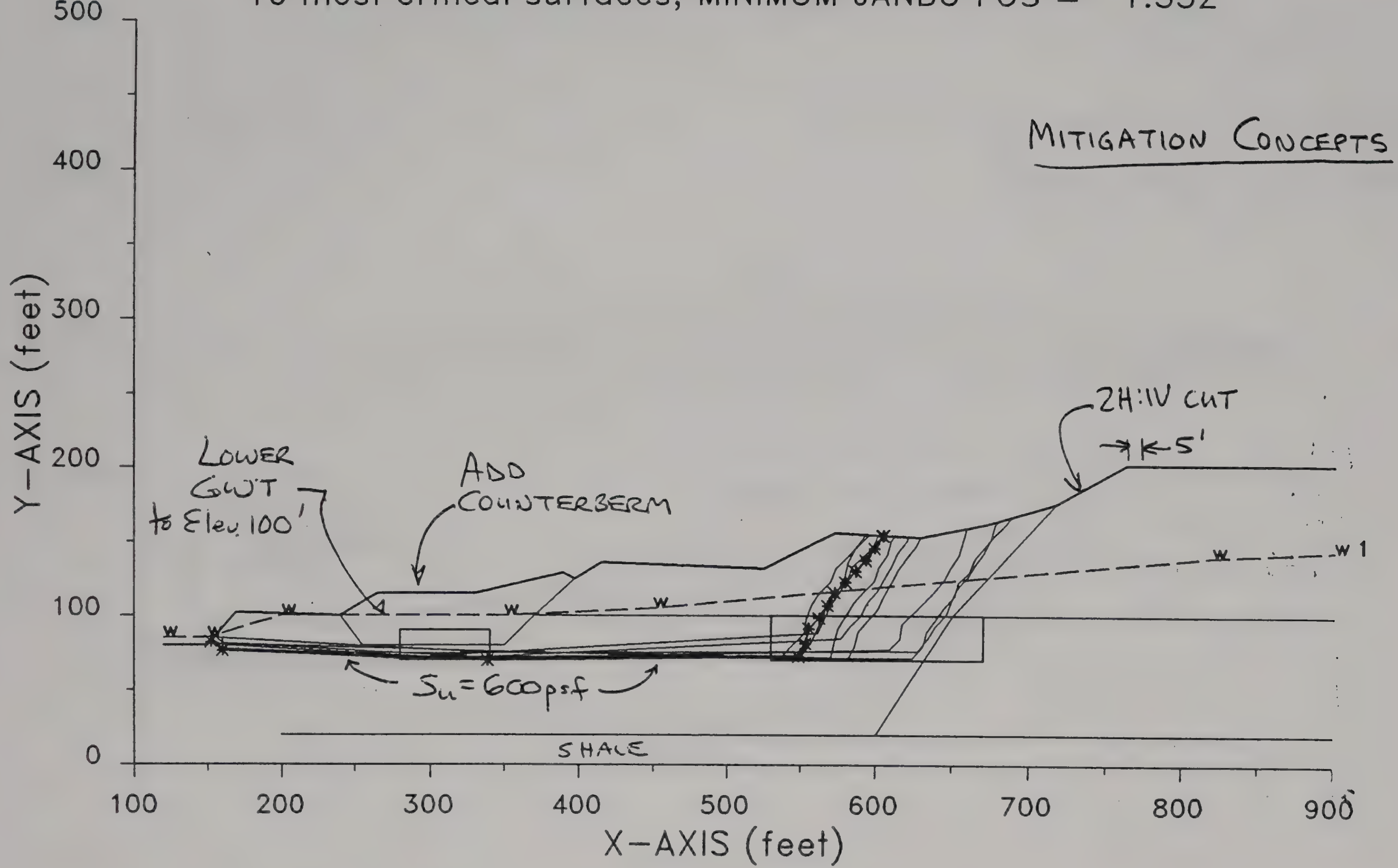




AF_2A 5-29-** 10:43
Normans Kill, total stress, gwt 140'
10 most critical surfaces, MINIMUM JANBU FOS = 1.319
POST-SLIDE CONDITIONS
Y-AXIS (feet)
X-AXIS (feet)
0
100
200
300
400
500
600
700
800
900
Su=1000 psf
Su=1000 psf
Across CLAY varves
Su=800 psf
Su=2,500 psf
W
W 1
Su=600 psf
Su=600 psf
Along CLAY VARVES
Su=3,000 psf
Su=6,000 psf SHALE

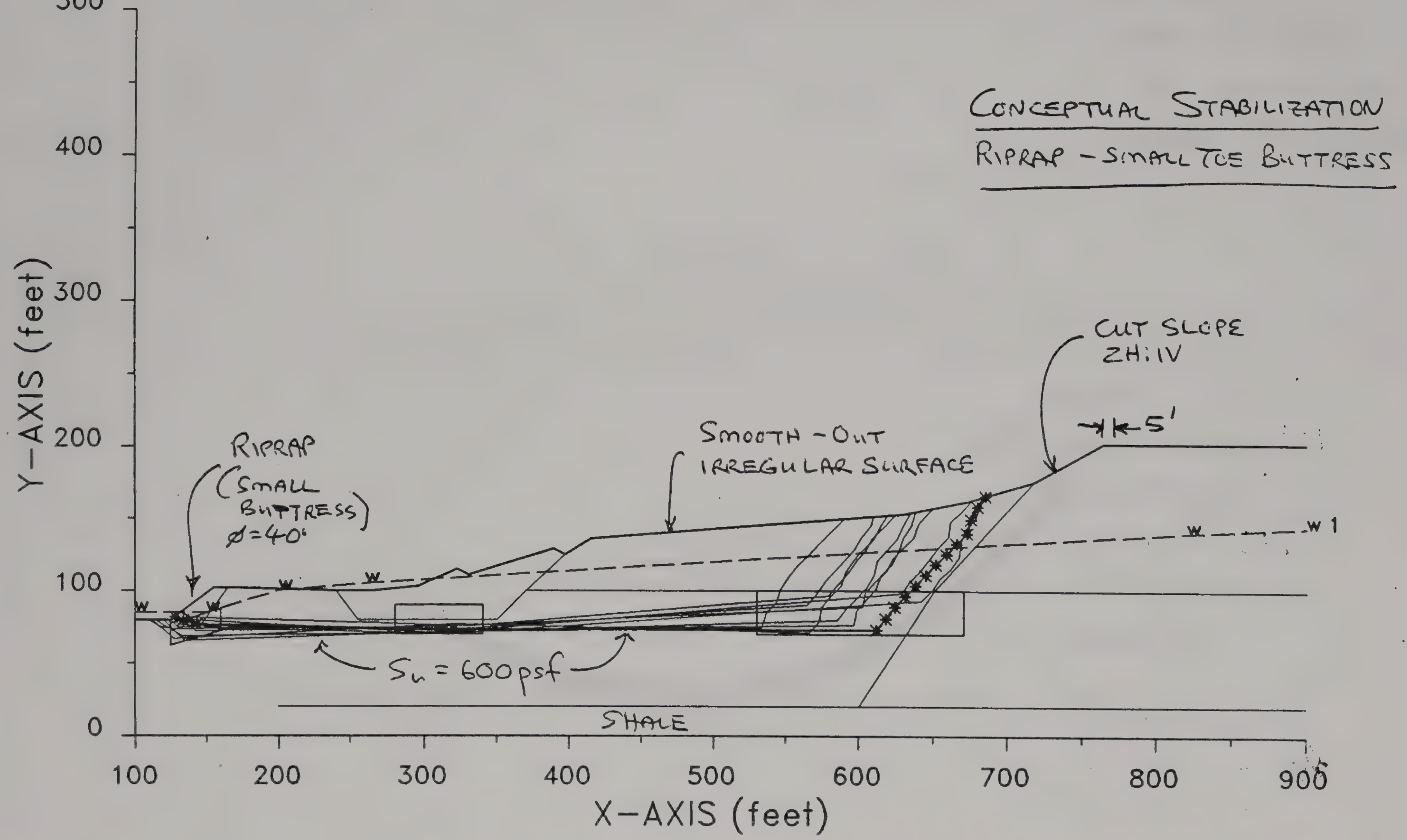
ST_2A 5-29-** 15:39

Normans Kill, total stress, gwt 140'
10 most critical surfaces, MINIMUM JANBU FOS = 1.332



ST_5B 5-29-** 17:35

Normans Kill, total stress, gwt 140'
10 most critical surfaces, MINIMUM JANBU FOS = 1.408

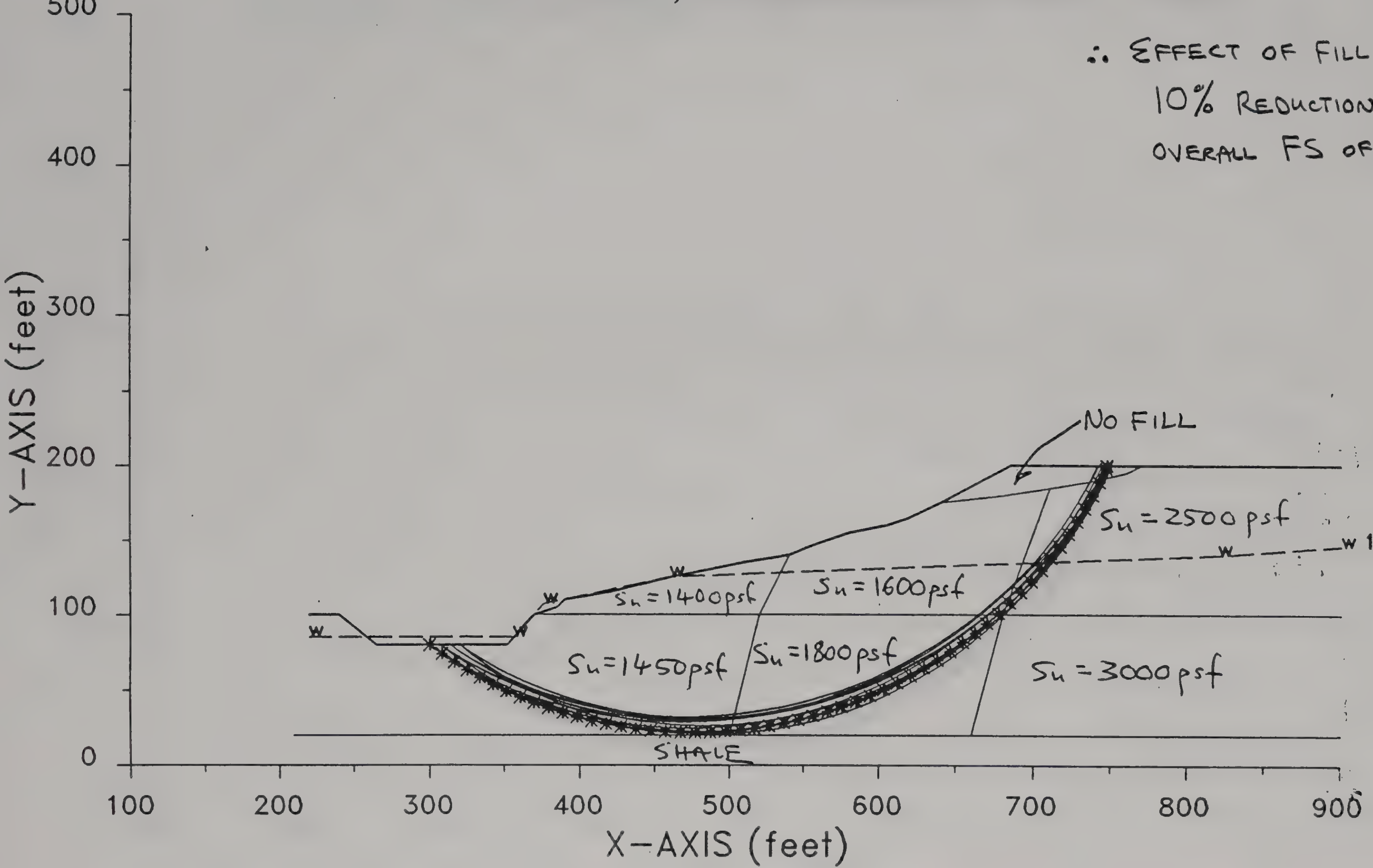


I_2B2 5-29-** 13:29

Normans Kill Slide, Initiate, no fill

10 most critical surfaces, MINIMUM BISHOP FOS = 1.121

∴ Effect of fill ~ 10% reduction in overall FS of slope.



I_2B 5-29-** 12:45

Normans Kill Slide, Initiate, surch
10 most critical surfaces, MINIMUM BISHOP FOS = 1.002